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JULY, 1955. **Vol. L, No. 7**

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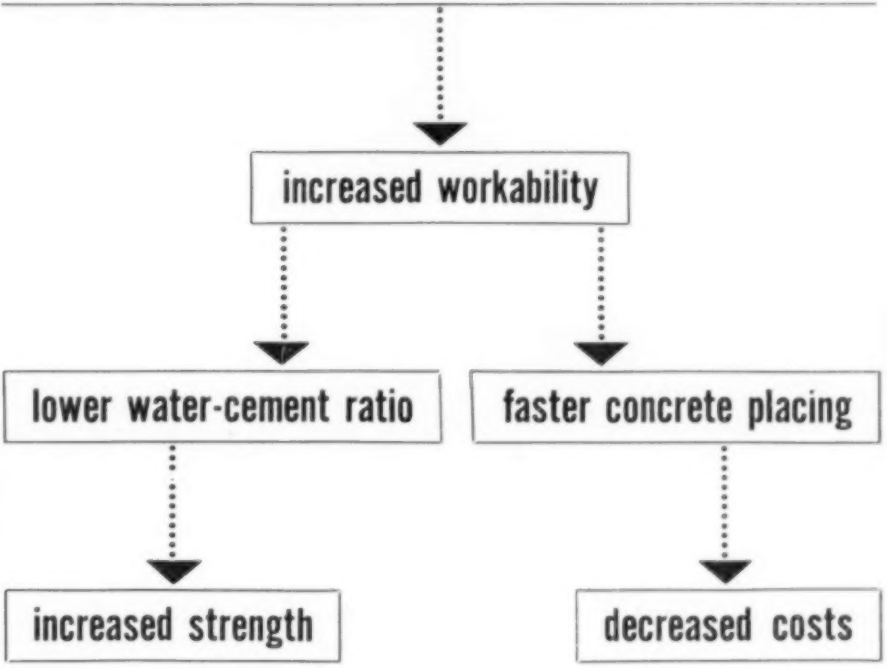

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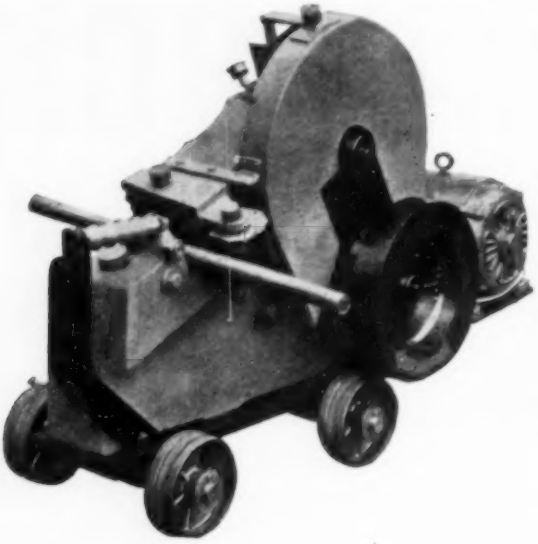

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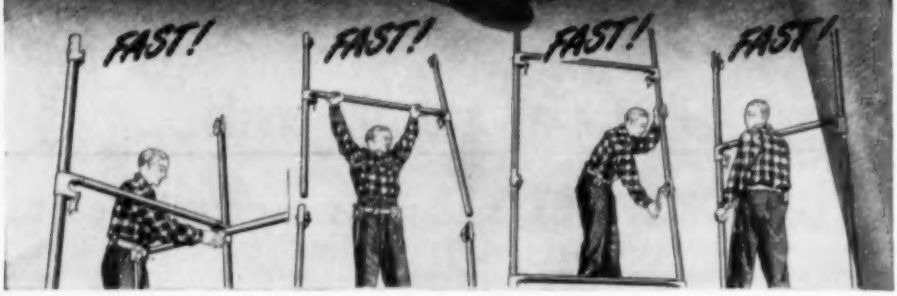

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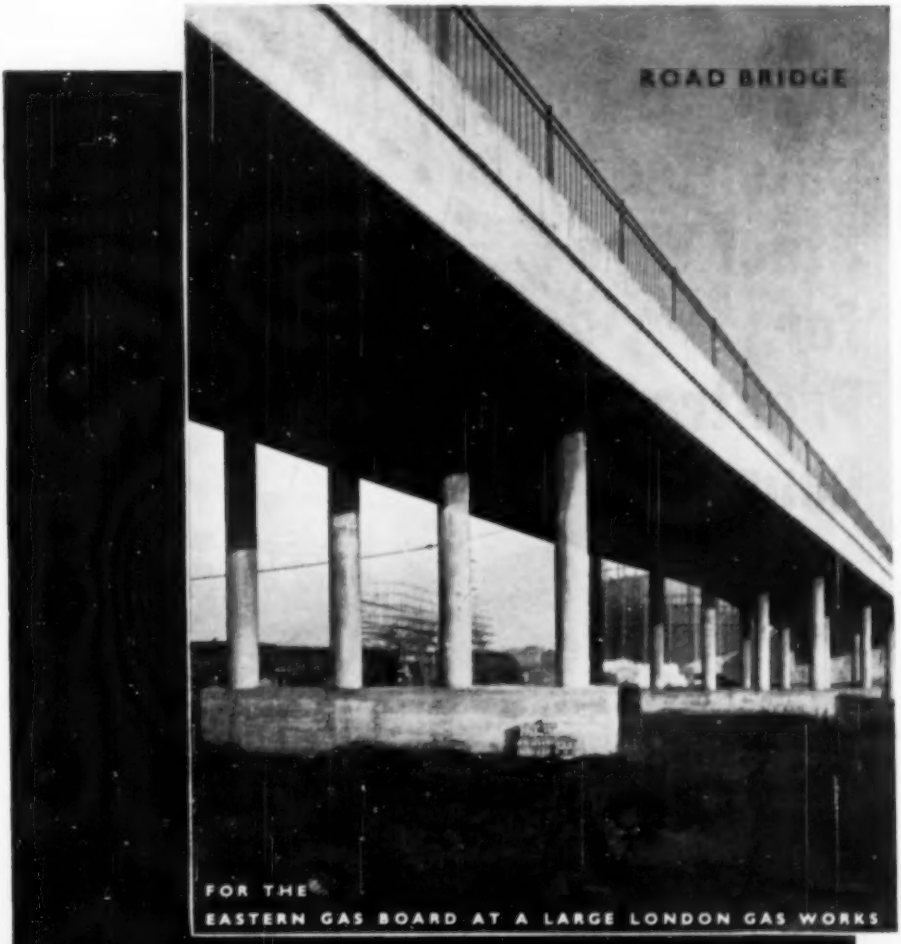

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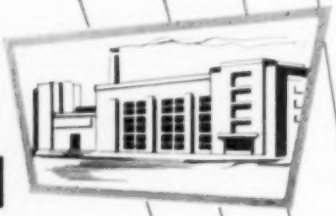

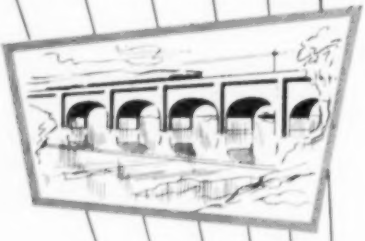

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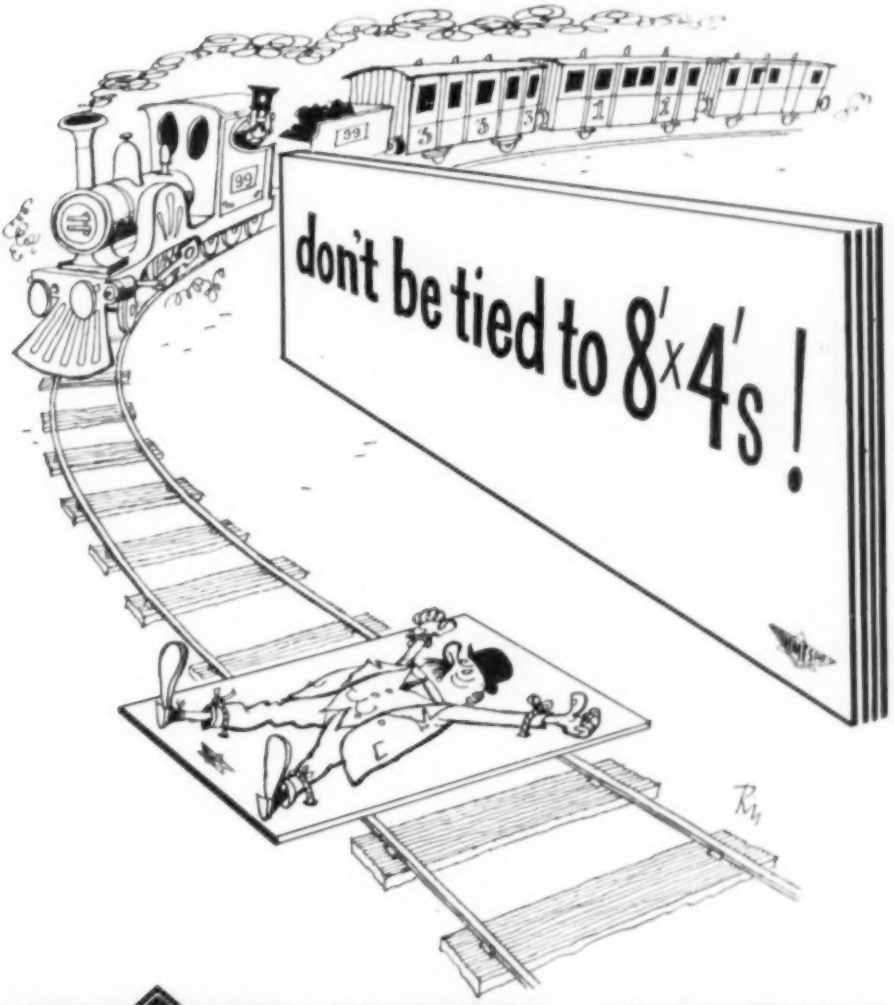

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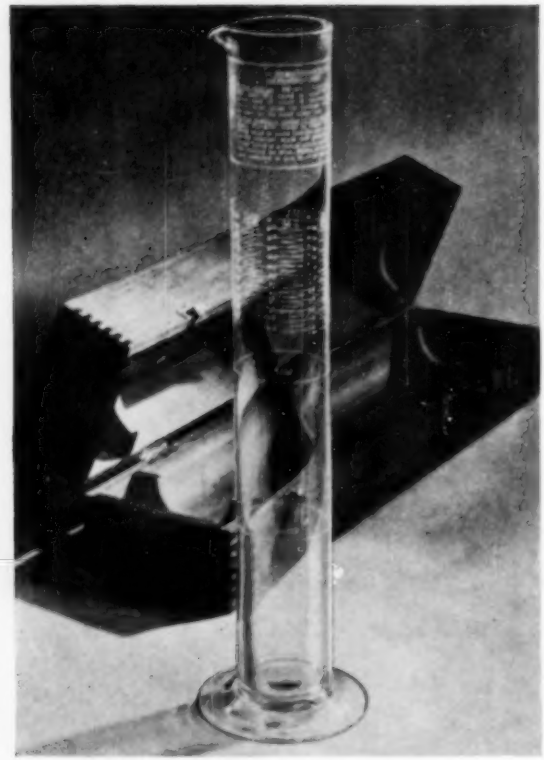

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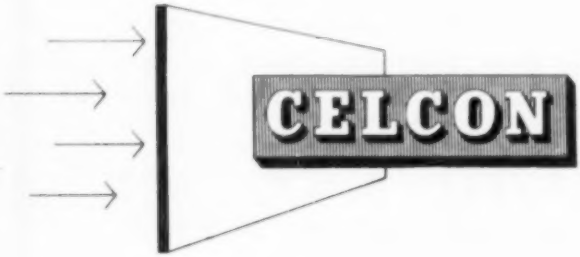

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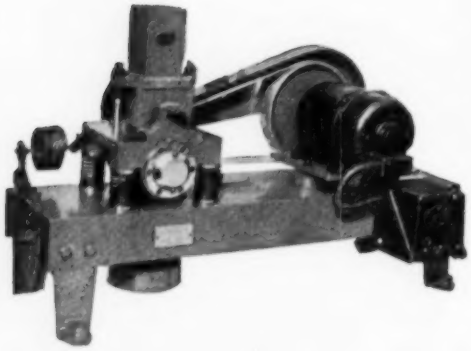

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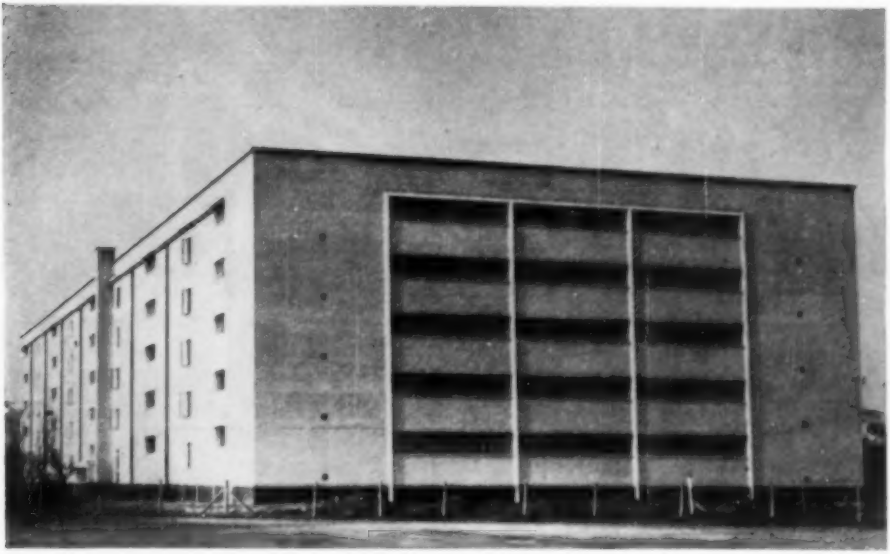

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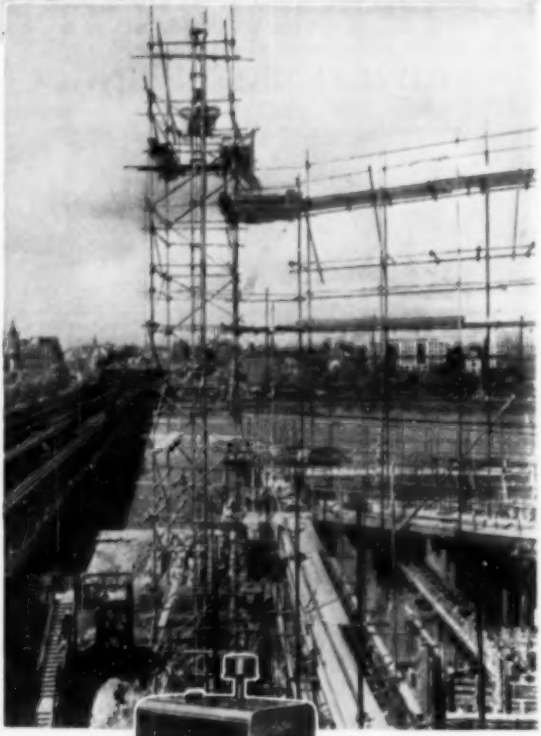

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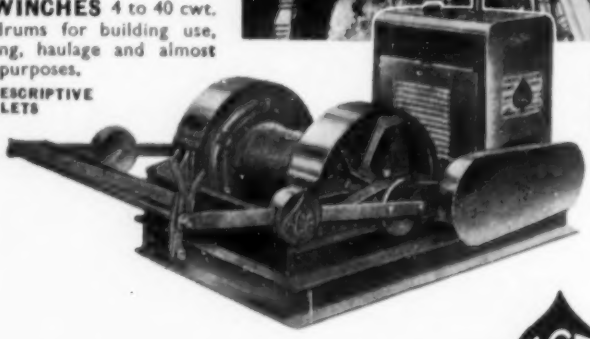

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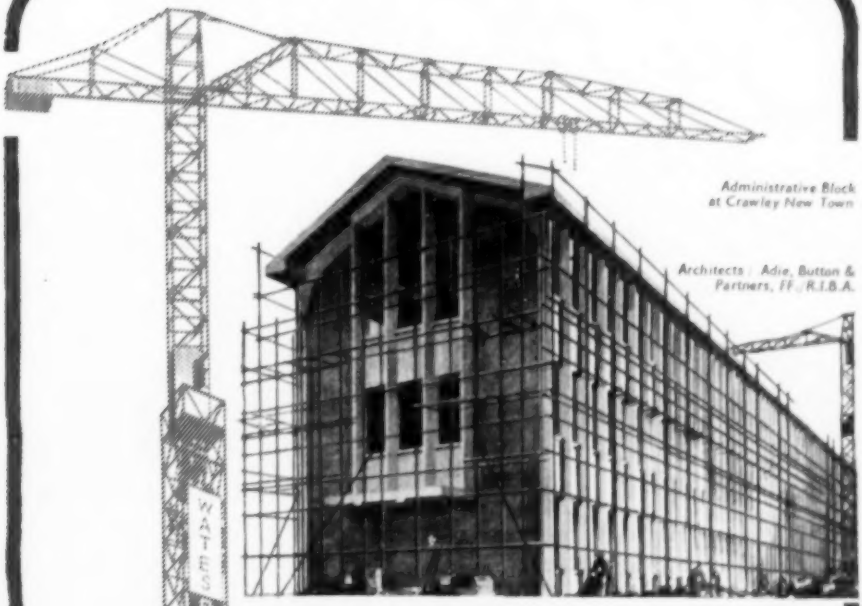

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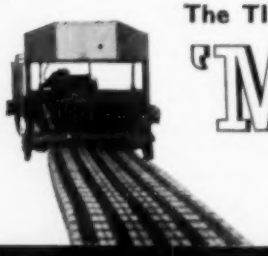

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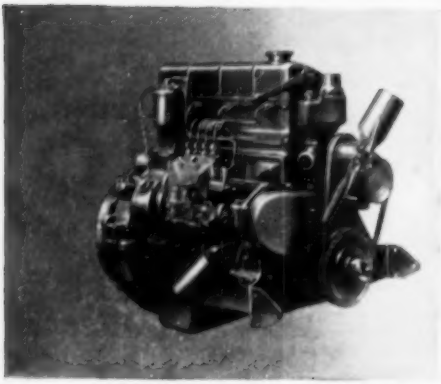


◀ 954E SERIES

BORE M.M. 100 · STROKE M.M. 115
NO. OF CYLINDERS 4
COMPRESSION RATIO 16:1
SWEPT VOLUME 3·61 LTRS. · CYCLES 4
ENGINE HEIGHT 32·62 INS.
ENGINE WIDTH 23·6 INS.
ENGINE LENGTH 33·83 INS.

B.H.P.	R.P.M.					
	1000	1200	1500	1600	1700	1800
1 HOUR RATE	26·5	32·0	38·5	40·5	42·0	43·0
12 HOUR RATE	23·0	28·0	34·0	35·75	37·0	38·0
24 HOUR RATE	21·0	25·6	31·0	32·5	33·6	34·5

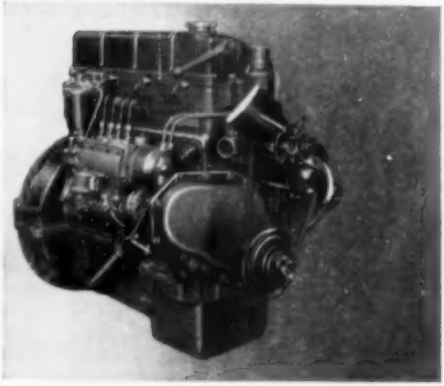
Conforming to the requirements of BSS.649/49

563E SERIES ▶

BORE M.M. 100 · STROKE M.M. 115
NO. OF CYLINDERS 4
COMPRESSION RATIO 16:1
SWEPT VOLUME 3·61 LTRS. · CYCLES 4
ENGINE HEIGHT 34·5 INS.
ENGINE WIDTH 26·88 INS.
ENGINE LENGTH 31·20 INS.

B.H.P.	R.P.M.					
	1250	1500	1800	2000	2250	2400
1 HOUR RATE	34·75	41·5	50·0	54·5	58·0	59·0
12 HOUR RATE	31·0	37·5	44·5	49·0	52·5	—
24 HOUR RATE	27·8	33·2	40·0	43·6	46·5	—

Conforming to the requirements of BSS.649/49



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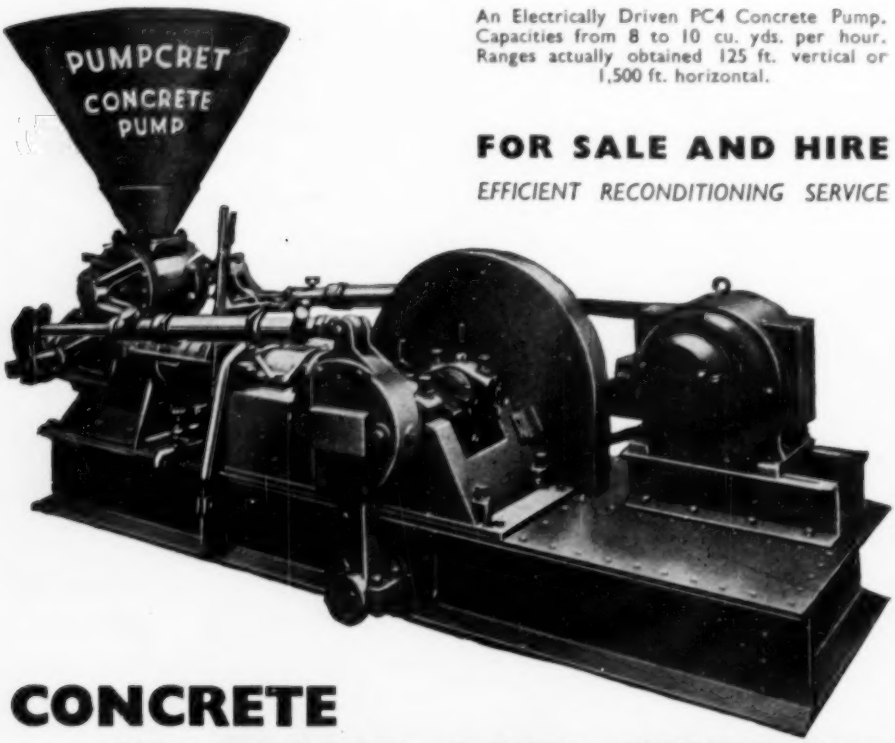

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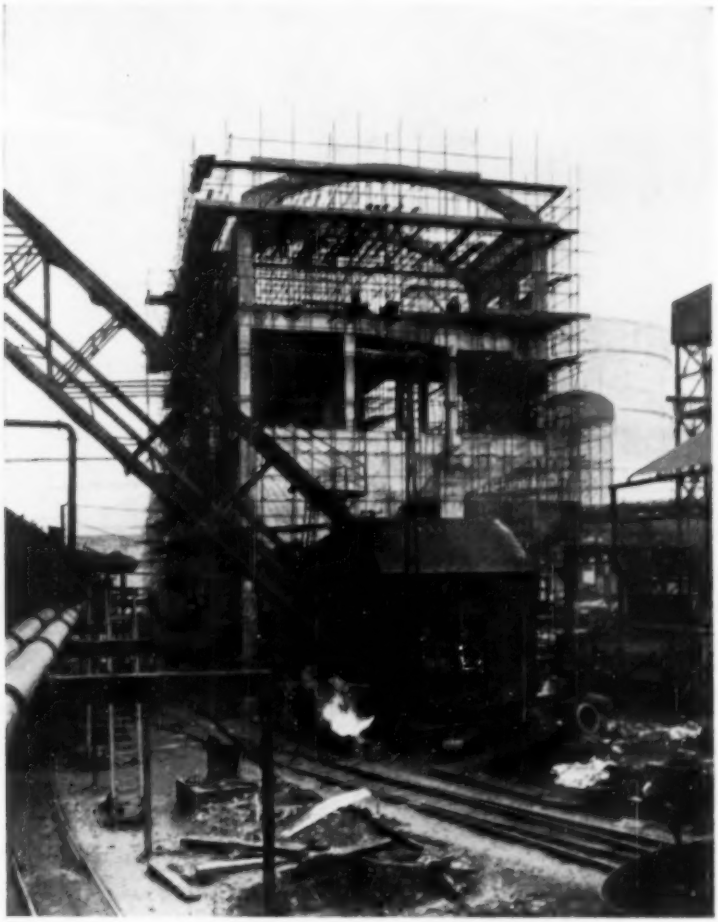

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Photo by courtesy of Lancashire Steel Corporation, Ltd.

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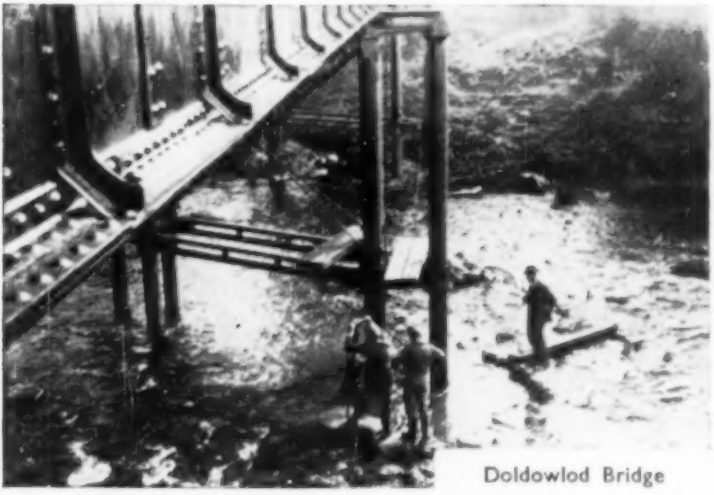

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CHIEF ENGINEER
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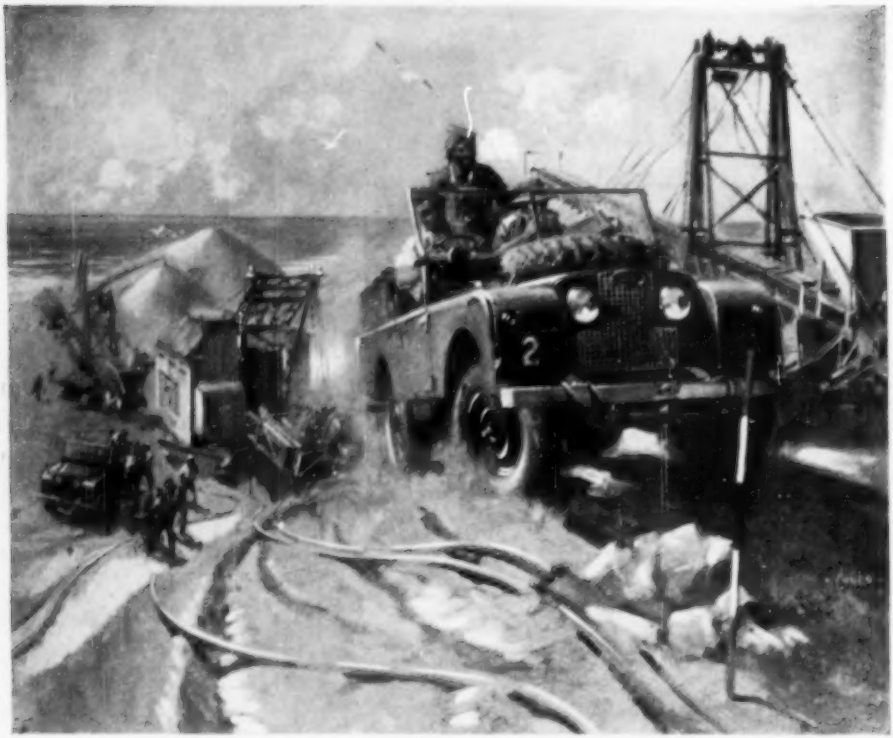

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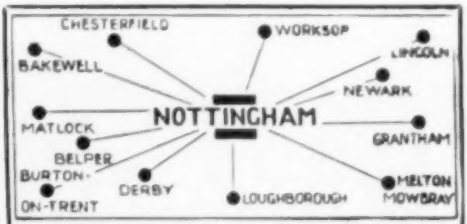

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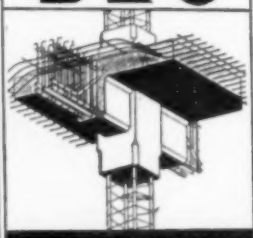

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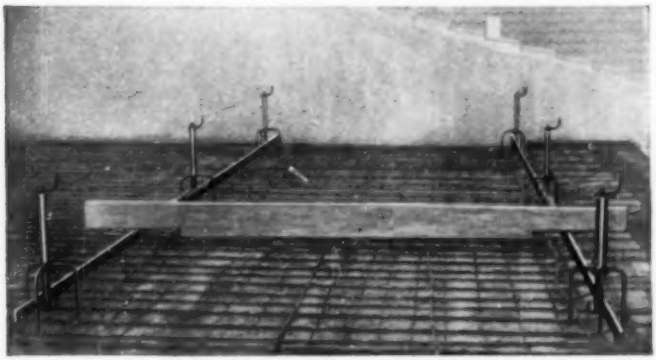

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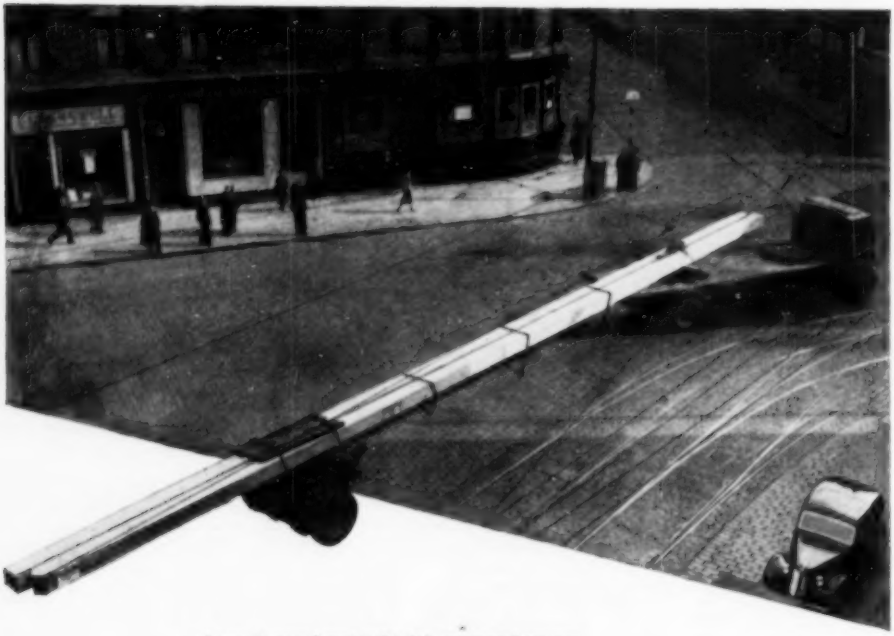

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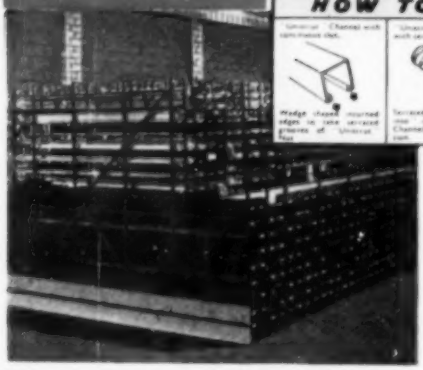

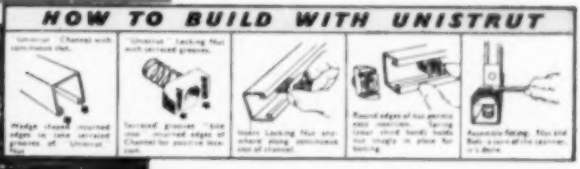

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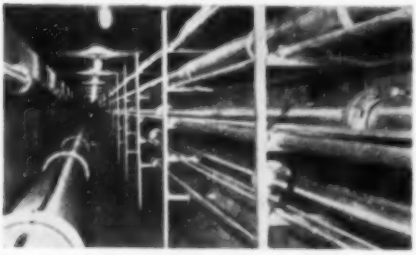

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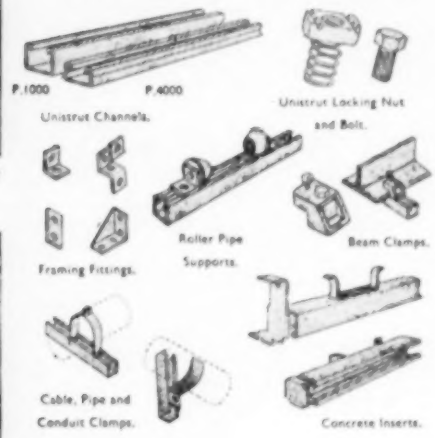

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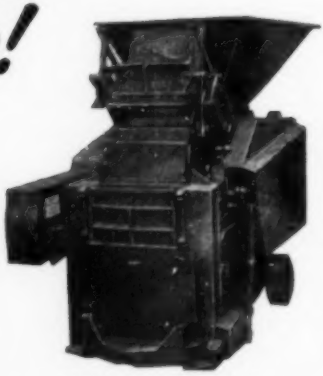

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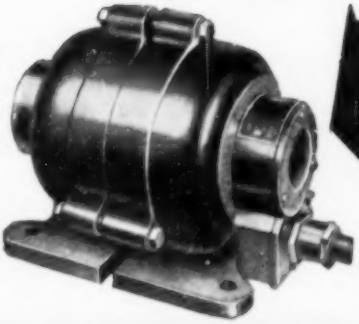

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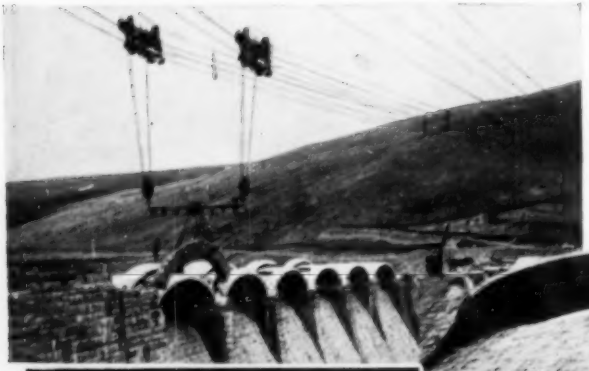

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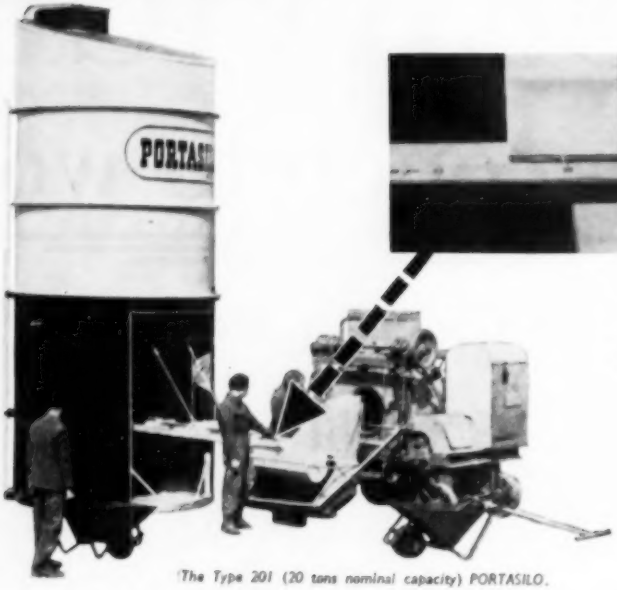

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Volume L, No. 7. LONDON, JULY, 1955.

EDITORIAL NOTES

Improvers for Concrete.

ONE of the earliest references to the use of a substance for improving the properties of concrete is the recommendation of Vitruvius nearly 2000 years ago that hog's fat be used to improve the watertightness of concrete floors. Since then scores of substances have been advocated for improving various properties of concrete, and many of these are, like Vitruvius's, fatty or soapy substances. Some of the improvers are ordinary commercial products that have been found to be beneficial to concrete and that have been sold under trade names at many times the commercial price. For example, in the early days of this century hydrated lime was sold under a trade name as a waterproofer at many times the commercial price, and much improved the concrete with which it was incorporated. The lime, of course, acted as a lubricant and improved workability, but the greatest benefit resulted from the fact that, having paid a high price for a small quantity of white powder which he was told was efficaceous only if thoroughly distributed throughout the mixture, the builder took pains to see that the concrete was thoroughly mixed, and the better mixing plus the improved workability produced a denser and more watertight concrete. Other examples were the sale at high prices of cheap commodities such as calcium chloride for quick hardening and waterglass for waterproofing and hardening concrete surfaces. This method of making profit from a discovery is not confined to the concrete industry.

With the extension of knowledge of the properties of these "improvers" it is now common practice for the concrete maker to buy some of them commercially at a fraction of the cost of the proprietary solutions and add them to each batch of concrete. In some cases, where the demand is great enough, improvers are added by the cement maker, and by using cements with special properties obtained by added chemicals the user is saved trouble and is also assured of a uniform product. The use of special cements also overcomes the difficulty arising from the fact that some of the chemicals used to impart special properties to Portland cement have different effects on different cements. For example, to promote quick setting it is generally recommended that the proportion of calcium chloride should be 1 to 50 parts of cement by weight, but it has been found that the proportion that gives the best and cheapest result differs with nearly every brand of cement, and the cement maker can be relied upon to

add the proportion that gives the most satisfactory results with the cement made at any particular works. In addition to Portland cements to which calcium chloride is added to promote quick setting, cements are now obtainable in Great Britain containing pozzolanic material to promote slow-hardening and thus reduce the heat of hydration and also to resist the attack of sulphates, soapy substances to improve impermeability and to promote water-repellency, and resinous substances to promote the workability of mortar. By additions to cement and concrete it is now possible to hasten or delay the setting of concrete, to increase workability, watertightness, and resistance to frost, to control the heat of hydration, to reduce segregation, to form colloidal grout, to make concrete that will expand on setting, and to produce concrete with a density as low as 20 lb. per cubic foot. In many cases more watertight and durable concrete can be obtained by the use of richer mixtures, but this is not always so because very rich mixtures may also have undesirable properties such as excessive shrinkage.

A notable addition to the list of concrete improvers was introduced in the U.S.A. some twenty years ago in the form of resins from wood which, after treatment to make them soluble, cause the formation in concrete of innumerable tiny bubbles of air. This addition reduces the density and the compressive strength of concrete but at the same time increases its durability when exposed to icy conditions and subjected to applications of salt. To some extent air-entrained concrete contradicts the theory that the denser a concrete and the higher its compressive strength the better will be all the other properties that are desirable in exposed concrete. Air-entrained concrete was first used for concrete roads in the U.S.A. because it reduced the deterioration of the surface when salts were applied to remove ice, but its use has since been extended for other purposes. The entrained air also acts as a lubricant or plasticiser, with the result that for structural work the same workability can be obtained with a lower water content and consequently without loss of compressive strength. Conversely, in the case of concrete made in machines and extruded from the mould immediately after it has been compressed, and which commonly contains insufficient moisture properly to hydrate the cement, it is found that a stronger product is obtained with the addition of an air-entraining agent because more water can be used. The reason for this is that air-entrained concrete made with a small water content, such as is the case with machine-made clinker blocks, has a rubbery nature and does not so easily fall apart when removed from the mould as would a similar concrete made with less water but without an air-entraining agent.

It is doubtful whether ordinary Portland cement made in accordance with standard specifications can in the foreseeable future be much further improved except by the addition of chemicals for specific purposes. Cement makers already produce cements for special purposes when the demand is sufficient for it to be economical to make special cements and provide separate storage for them. In cases where special properties are required for which special cements are not available the concrete maker has the choice of "improvers" that have stood the test of twenty years or so, but it must be borne in mind that in some cases the addition of a chemical to produce one desired result may adversely affect one or more of the other desirable properties of concrete.

Analysis of Single-bay Gable Frames.

By V. A. MORGAN, M.Eng., A.M.Inst.C.E.

THE frame shown in *Fig.* 1(*a*) is rigidly fixed at the column bases, and the rafters FB and FC are assumed to be the same length (S_o) and to have the same moments of inertia (I_o). Their stiffness ratios $\left(\frac{I_o}{S_o} = K_o\right)$ are assumed to be unity. Columns BA and CD are h ft. long and their stiffness ratios are K_1 and K_2 ($K_2 = nk_1$). The span (L ft.) is measured between the centre-lines of the columns, and the vertical rise of the ridge above the eaves is r ft. In this article the general solution is derived for the bending moments and thrusts at A, B, C, and D when a horizontal load P is applied at the eaves B.

In *Fig.* 1(*b*, *c* and *d*) it is shown that a bending moment M at B (the foot of the rafter-frame BFC) will cause a negative bending moment (anticlockwise

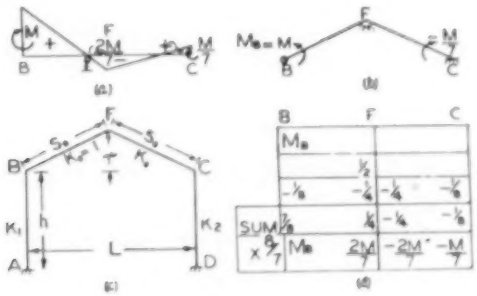


Fig. 1.

bending moments are considered to be positive) of $\frac{M}{7}$ at C if the frame is considered to be fixed at C; the carry-over factor from B to C is then $-\frac{1}{7}$.

To obtain the stiffness ratios of B and C in the frame BFC a unit rotation is applied at B and the resulting bending moment at B gives the stiffness ratio. Since the rotation at B is $\frac{1}{EI_o}$ times the area of the bending-moment diagrams between B and C when C is considered to be fixed, this is

$$\frac{8M}{7} \times \frac{S_o}{EI_o} = \left(\frac{4M}{7} + \frac{2M}{7}\right)\frac{S_o}{7EI_o},$$ so that, equating this to unity, the bending moment at B due to unit rotation is $\frac{7EI_o}{2S_o} = \frac{7EK_o}{2}$. As the stiffness ratio for column BA is $4EK_1$ if it is fixed at the base, or $3EK_1$ if it is hinged at the base, and as K_o equals unity, the stiffness ratios of the frame BFC to column BA at B are as 7 is to $8K_1$ for fixed bases and as 7 is to $6K_1$ for hinged bases.

The expression for the thrust of the rafter ($H_{ra} = H_o + H_m$) is obtained in a similar manner to the equation for a parabolic-arch frame, and comprises

two terms, namely, the thrust H_o due to forces acting on the frame BFC when the feet of the rafter are considered to be pin-jointed, and the thrust H_m due to the bending moments acting at B and C. If the resultant thrust is negative then the feet of the rafter exert an inward thrust, and if the resultant thrust is positive an outward thrust is exerted.

If M_B and M_C are fixing moments at B and C it can be shown by the method of moment distribution that $M_{FB} = \frac{M_B - M_C}{4}$ and $M_{FC} = \frac{M_C - M_B}{4}$, and the reaction $R_B = \frac{-(M_B + M_C)}{L}$. Therefore, by calculating moments about F, $H_M = \frac{3}{4r}(M_B - M_C)$. Then $H_m = H_o + \frac{3}{4r}(M_B - M_C)$. To obtain H_o, the frame, which is considered to be hinged at B and C, is given at the hinge B a horizontal deflection (Δ) in an outward direction.

Then $\Delta = 2\int_0^{S_o} \frac{My\,ds}{EI_o}$, but as $M = H_o y$, $\Delta = 2\int_0^{S_o} \frac{H_o y^2 ds}{EI_o} = \frac{2H_o S_o^3}{3EI_o}\sin^2\theta$.

Since $y = S\sin\theta$, and as $S_o \sin\theta = r$,

$$\Delta = \frac{2H_o r^2 S_o}{3EI_o} = \frac{2H_o r^2}{3EK_o}.$$

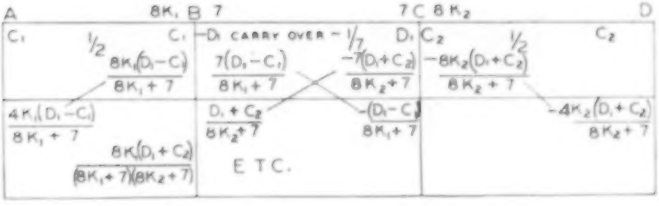
Therefore $H_o = \frac{3EK_o\Delta}{2r^2}$. If both hinges move outwardly, $H_o = \frac{3EK_o}{2r^2}(\Delta_1 + \Delta_2)$.

By the method of slope deflection it can be shown that M_{BA}; the fixing moment at B due to an outward horizontal movement of Δ is $\frac{6EI_o}{S_o^2}\Delta_{FB}$ where Δ_{FB} is the deflection of F relative to B at right-angles to FB. It can also be shown that M_{BA} is equal to $\frac{S_o\Delta}{2r}$ by drawing a deflection triangle with Δ parallel to BC and with Δ_{FB} perpendicular to FB and FC for their relative deflections. Since the deflection of F will be small its deflected position may be considered to be at right-angles to both FB and FC and $\frac{\Delta_{FB}}{\frac{1}{2}\Delta}$ will equal $\frac{S_o}{r}$, so that $\Delta_{FB} = \frac{S_o\Delta}{2r}$. Therefore the fixing moment $M_{BF} = \frac{3EK_o\Delta}{r} = \frac{3EK_o}{r}(\Delta_1 \pm \Delta_2)$, in which Δ_2 is positive if both feet of the rafter deflect outwardly and negative if they deflect inwardly. The fixing moment M_{CF} is equal to $-M_{BF}$.

From the foregoing expressions the thrusts and bending moments in the frame, due to a horizontal load applied at B, may be found. The load (P) causes horizontal deflections Δ_1 at B and Δ_2 at C which produce fixing moments in column C_1 equal to $\frac{6EK_1\Delta_1}{h}$ at A and B, and in column C_2 equal to $\frac{6EK_2\Delta_2}{h}$ at C and D.

At the feet of the rafter the fixing moments M_{BF} and M_{CF} are $-D_1$ and D_1 respectively where $D_1 = \frac{3(\Delta_1 - \Delta_2)EK_o}{r}$.

TABLE I.



The ratio of decrease of alternate distributions is $\frac{1}{(8K_1+7)(8K_2+7)}$, and the multiplier is $\frac{1}{1-\text{ratio}}$, or $\frac{(8K_1+7)(8K_2+7)}{8\alpha}$

where $\alpha = 8K_1K_2 + 7K_1 + 7K_2 + 6$ (see *Table* 1).

Substituting for A_1 and A_2, D_1 becomes $\frac{h}{2r}\left(\frac{C_1}{K_1} - \frac{C_2}{K_2}\right)$, and, by summing the moment-distribution terms to infinity,

$$2\alpha . M_{AB} = C_1\left[\alpha + 7K_2 + 6 + \frac{h}{r}(4K_2 + 3)\right] + C_2\left[K_1 + \frac{hK_1}{rK_2}(4K_2 + 3)\right] \quad . \quad (1)$$

$$\alpha . M_{BA} = C_1\left[7K_2 + 6 + \frac{h}{r}(4K_2 + 3)\right] - C_2\left[K_1 + \frac{hK_1}{rK_2}(4K_2 + 3)\right] . \quad . \quad (2)$$

$$\alpha . M_{CD} = C_2\left[7K_1 + 6 + \frac{h}{r}(4K_1 + 3)\right] - C_1\left[K_2 + \frac{h}{r}(4K_1 + 3)\right] \quad . \quad . \quad (3)$$

$$2\alpha . M_{DC} = C_2\left[\alpha + 7K_1 + 6 + \frac{h}{r}(4K_1 + 3)\right] + C_1\left[K_2 + \frac{h}{r}(4K_1 + 3)\right] \quad . \quad (4)$$

Also $H_1 = \frac{M_{AB} + M_{BA}}{h}$ and $H_2 = \frac{M_{CD} + M_{DC}}{h}$.

$$H_{ra} = H_o + \frac{3}{4r}(M_{BC} - M_{CB}) = -\frac{D_1}{2r} - \frac{3}{4r}(M_{BA} - M_{CD})$$

$$2r . \alpha . H_{ra} = C_1\left[12K_2 + 9 + \frac{h}{rK_1}(2\alpha - 6K_2 - 6K_1 - 9)\right]$$
$$- C_2\left[12K_1 + 9 + \frac{h}{rK_2}(2\alpha - 6K_1 - 6K_2 - 9)\right].$$

By equating H_{ra} to H_2, C_2 is obtained in terms of C_1, and by substituting for C_2 in $P = H_1 + H_2$,

$$\frac{2C_1}{hP} = \frac{r(\alpha + 21K_1 + 18) + 6h(4K_1 + 3) + \frac{h^2}{rK_2}(2\alpha - 6K_1 - 6K_2 - 9)}{r(\alpha - 6K_1K_2 + 18) + 6h(K_2 + K_1 + 6) + \frac{h^2}{2rK_2}\left(\alpha + 8K_2^2 + \frac{6K_2}{K_1} + 37K_2 + \frac{4K_2^2}{K_1} - 3K_1\right)} \quad . \quad (5)$$

Let β represent the denominator on the right-hand side of equation (5). Then

$$\frac{2\beta C_2}{hP} = 3K_2r + 3h\left(8K_2 + \frac{3K_2}{K_1} + 3\right) + \frac{h^2}{rK_1}(2\alpha - 6K_1 - 6K_2 - 9) \quad . \quad (6)$$

Substituting for C_1 and C_2 in equations (1) to (4),

$$\frac{2\beta}{hP}.M_{AB} = r(\alpha - 4K_1K_2 + 7K_1 + 18) + h(12K_1 + 2K_2 + 24) + \frac{h^2}{rK_2}(\alpha + 3K_2 - 3K_1 - 3) \quad . \quad (7)$$

$$\frac{2\beta}{hP}.M_{BA} = r(7K_2 + 24) + h(4K_2 + 30) + \frac{3h^2}{rK_2}(4K_2 + 1). \quad . \quad . \quad . \quad (8)$$

$$\frac{2\beta}{hP}.M_{CD} = -rK_2 + h\left(9 - 4K_2 - \frac{3K_2}{K_1}\right) + \frac{3h^2}{rK_1}(4K_1 + 1) \quad . \quad . \quad . \quad (9)$$

$$\frac{2\beta}{hP}.M_{DC} = rK_2 + h\left(10K_2 + \frac{3K_2}{K_1} + 9\right) + \frac{h^2}{rK_1}(\alpha - 3K_2 + 3K_1 - 3) \quad . \quad (10)$$

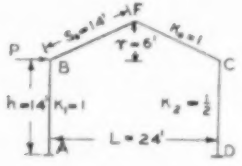


Fig. 2.

Equations (5) to (10) apply to all types of fixed-base single-bay frames with columns of equal or unequal stiffness ratios when a horizontal load is applied at the eaves. The bending moments are found by substituting numbers for r, K_1, K_2, and h. As $hH_1 = M_{AB} + M_{BA}$, the thrusts may be found. The vertical reactions at A and D are found from $P \times h - M_{AB} - M_{DC} = V_D \times L = -V_A \times L$, and the bending moment at the ridge is obtained by calculating the moments of all external forces to either the left or right of F.

The analysis of the frame with any system of loading can now be investigated. For the purpose of illustration, a frame in which column BA is assumed to have double the stiffness of column CD is selected (*Fig.* 2). The following sizes and values are assumed.

S_o, 14 ft.; h, 14 ft.; L, 24 ft., $r\left(\frac{L}{4}\right)$, 6 ft.; I_o, 568-in.^{4}; $K_o = \left(\frac{I_o}{S_o}\right)$, which is assumed to be unity; I_1, 568-in.^{4}; I_2, 284-in.^{4}, so that K_1 is unity, and K_2 is 0·5. Substituting these values in equations (5) and (6) the following are obtained. $\frac{6C_1\beta}{hP} = 7343$; $\frac{3C_2\beta}{hP} = 1676$; and, as $\Delta_1 = \frac{C_1h}{6EK_1}$ and $\Delta_2 = \frac{C_2h}{6EK_2}$, we have $\Delta_1 = \frac{7343Ph^2}{36E\beta}$ and $\Delta_2 = \frac{1676Ph^2}{9E\beta}$ where $\beta = 2215$.

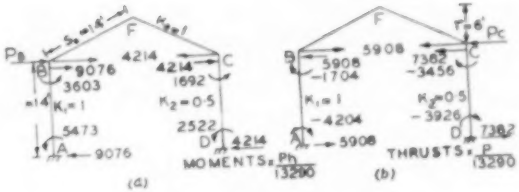


Fig. 3.

Similarly substituting in equations (7) to (10),

$$M_{AB} = 5473.\frac{Ph}{6\beta};\ M_{BA} = 3603.\frac{Ph}{6\beta};\ M_{CD} = 1692.\frac{Ph}{6\beta};\ M_{DC} = 2522.\frac{Ph}{6\beta}.$$

Therefore $H_1 = \dfrac{9076P}{6\beta}$ and $H_2 = \dfrac{4214P}{6\beta}$.

The coefficients of these values are indicated in *Fig.* 3(*a*).

The thrusts and bending moments due to the load P acting at C are obtained by substituting 0·5 for K_1 and 1 for K_2 in equations (5) to (10), but, as the load acts on the opposite side of the frame, M_{AB} becomes M_{DC} and similarly all other bending moments and thrusts change position. The bending moments and thrusts will be

$$M_{CD} = -3456\frac{Ph}{6\beta};\ M_{DC} = -3926\frac{Ph}{6\beta};\ M_{BA} = -1704\frac{Ph}{6\beta};$$

$$M_{AB} = -4204\frac{Ph}{6\beta};\ H_1 = 5908\frac{P}{6\beta};\ H_2 = 7382\frac{P}{6\beta}.$$

The coefficients of these values are shown in *Fig.* 3(*b*).

Also $\dfrac{6C_2\beta}{hP} = 4396$ and $\dfrac{3C_1\beta}{hP} = 3352$, and, since $\Delta_2 = \dfrac{C_2h}{6EK_2}$ and $\Delta_1 = \dfrac{C_1h}{6EK_1}$,

$\Delta_2 = \dfrac{4396Ph^2}{18E\beta}$ and $\Delta_1 = \dfrac{1676Ph^2}{9E\beta}$. It is seen that Δ_2 for P acting at B equals Δ_1 for P acting at C and is in accord with Maxwell's theorem of reciprocal deflections.

Application of these results consists in counterbalancing the thrusts at B and C set up by the loadings.

Uniform Load on Span.

Considering the case where a uniform load (wL) acts on the horizontal projection of the rafters, the fixing moment at B caused by the load $\dfrac{wL}{2}$ on BF is M and equals $\dfrac{wL}{48}$, and the fixing moment at C due to the load on CF is $-M$ and is equal to $-\dfrac{wL}{48}$. The fixing moments at F due to this load when B and C

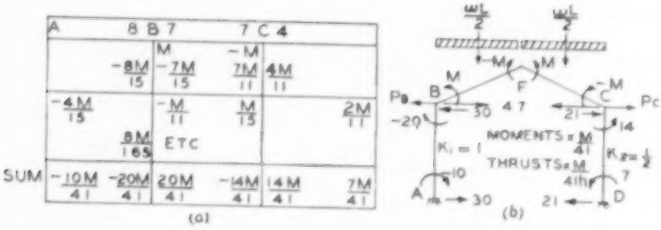


Fig. 4.

are considered to be hinged are $\frac{-3M}{2}$ for FB and $\frac{3M}{2}$ for FC and, by taking moments about F, $r.H_0 = \frac{-wL^2}{8} - \frac{3M}{2} = \frac{-15M}{2}$.

After distributing the fixing moments (*Fig.* 4*a*), the unbalanced thrusts are calculated from the thrusts H_1 and H_2 of the rafter.

$$H_{ra} = H_0 + \frac{3}{4r}(M_{BF} - M_{CF}) = \frac{-15M}{2r} + \frac{51M}{2r \times 41} = \frac{-47M}{41}.$$

$$H_1 = \frac{30M}{574};\ H_2 = \frac{21M}{574},$$

so that $\frac{P_B}{M} = \frac{-30}{574} + \frac{47}{41} = \frac{628}{574}$ and the balancing force to this acts to the right of B.

Similarly $\frac{P_C}{M} = \frac{-21}{574} + \frac{47}{41} = \frac{637}{574}$, and the balancing force acts to the left of B. Components of $\frac{P_B}{M}$ acting in the opposite direction to those shown in *Fig.* 3(*a*) and of $\frac{P_C}{M}$ acting in the opposite direction to those shown in *Fig.* 3(*b*) are added algebraically to the thrusts and moments shown in *Fig.* 4. Thus

$\frac{H_1}{M_{(final)}} = \frac{30}{574} + \frac{628}{574} \times \frac{9076}{13,290} - \frac{637}{574} \times \frac{5908}{13,290} = \frac{678}{2215}$ acting to the right at A;

$\frac{H_2}{M_{(final)}} = \frac{21}{574} - \frac{628}{574} \times \frac{4214}{13,290} + \frac{637}{574} \times \frac{7382}{13,290} = \frac{678}{2215}$ acting to the left at B;

$\frac{M_{BA}}{M} = \frac{-20}{41} - \frac{628}{574} \times \frac{3603h}{13,290} + \frac{637}{574} \times \frac{1704h}{13,290} = \frac{-5866}{2215}$.

Similarly $M_{AB} = \frac{-3626M}{2215}$, $M_{CD} = \frac{5386M}{2215}$, and $M_{BC} = \frac{4106M}{2215}$, and as $M = \frac{wL^2}{48}$ these bending moments and thrusts may be determined. The bending moments and reactions at F and at any section may be determined by statics.

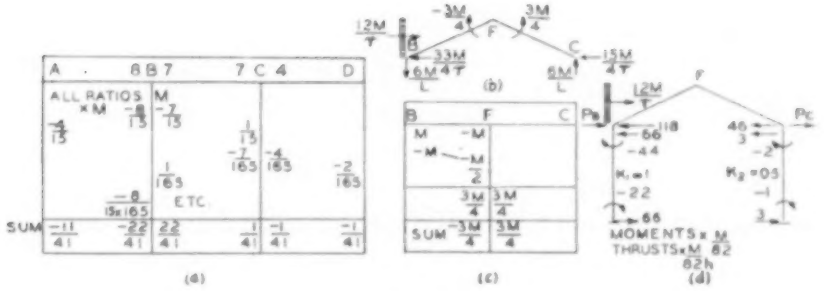


Fig. 5.

Uniform Horizontal Load.

A uniform horizontal wind load on the vertical projection of the roof causes fixing moments $M = \frac{pr}{12}$ at B and $-M$ at F where p is the load per foot run of the vertical projection r. It is first necessary to investigate the frame BFC assuming that it is pin-jointed at B and C. *Fig.* 5(*c*) shows that $M_{FB} = \frac{-3M}{4}$ and $M_{FC} = \frac{3M}{4}$, and, by calculating moments about F, $H_{oB} = \frac{+33M}{4r}$ and $H_{oC} = \frac{-15M}{4r}$. Substituting the distributed values from *Fig.* 5(*a*) in the thrust equation,

$$H = H_o + \frac{3}{4r}(M_B - M_C); \quad H_{BC} = \frac{+33M}{4r} + \frac{63M}{164r} = \frac{59M}{41}; \text{ and}$$

$$H_{CB} = \frac{-15M}{4r} + \frac{63M}{164r} = \frac{23M}{41}; H_1 = \frac{33}{41h}; H_2 = \frac{3}{82h}; P_B = \frac{59M}{41} + \frac{33M}{574} = \frac{1718M}{1148}$$

acting to the right at B. $P_C = \frac{3M}{1148} + \frac{23M}{41} = \frac{647M}{1148}$ acting to the right at C.

Adding the components of P_B shown in *Fig.* 3(*a*) and the components of P_C shown in *Fig.* 3(*b*) to the unbalanced forces and bending moments in *Fig.* 5(*d*),

$$\frac{H_1}{M}_{(\text{final})} = \frac{66}{1148} - \frac{1718}{1148} \times \frac{9076}{13,290} - \frac{647}{1148} \times \frac{5908}{13,290} = \frac{16,148}{13,290}.$$

$$\frac{H_2}{M}_{(\text{final})} = \frac{3}{1148} - \frac{647}{1148} + \frac{3456}{13,290} - \frac{1718}{1148} \times \frac{4214}{13,290} = \frac{10,432}{13,290}.$$

$$\frac{M_{BA}}{M}_{(\text{final})} = \frac{-22}{41} + \frac{3603}{13,290} \times \frac{1718 \times h}{1148} + \frac{1704}{13,290} \times \frac{647h}{1148} = \frac{27,267}{4430}.$$

Similarly final values of M_{CD}, M_{AB} and M_{DC} are $\frac{20,798M}{4430}$, $\frac{144,271M}{13,290}$, and $\frac{83,654M}{13,290}$.

If a suction of $\frac{12M}{r}$ is applied to the horizontal projection of the rafter FC then

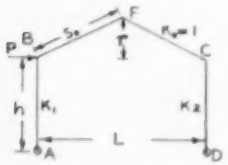


Fig. 6.

TABLE II.

6K1 B 7	−1/7	7 C 6K2
1 C1	−D1	D1 C2
6K1(D1 − C1) / 6K1 + 7	7(D1 − C1) / 6K1 + 7	−7(D1 + C2) / 6K2 + 7 — 6K2(D1 + C2) / 6K2 + 7
	ETC.	

the horizontal thrusts and bending moments, calculated in a similar manner, are

$$H_{1(\text{final})} = \frac{12{,}700M}{13{,}290}\ ;\ H_{2(\text{final})} = \frac{13{,}880M}{13{,}290}\ ;\ M_{CD} = \frac{21{,}275M}{4430}\ ;$$

$$M_{DC} = \frac{130{,}495M}{13{,}290}\ ;\ M_{BA} = \frac{26{,}490M}{4430},\ \text{and}\ M_{AB} = \frac{98{,}330M}{13{,}290}.$$

The difference between these values and those in the previous case is due to the difference in the stiffness of the two columns.

Single-gable Frame with Hinged Column-bases.

A frame having dimensions and stiffnesses as shown in *Fig.* 6 is free to rotate at the bases A and D. The stiffness ratios of the members joining at B and C are as 7 is to $6K_1$ for BF to BA and as 7 is to $6K_2$ for CF to CD. The values of the fixing moments C_1 and C_2 at the column heads are $C_1 = \frac{3EK_1\Delta_1}{h}$ and $C_2 = \frac{3EK_2\Delta_2}{h}$, so that D_1 (the fixing moment) is M_{CF} and equal to $\frac{h}{r}\left(\frac{C_1}{K_1} - \frac{C_2}{K_2}\right)K_o$, and $M_{BF} = -D_1$ and $H_o = \frac{h}{2r^2}\left(\frac{C_1}{K_1} - \frac{C_2}{K_2}\right)K_o$. (See *Table* II.)

Fixing moments are distributed in the same way as in frames with fixed bases. H_2 is equated to H_{ra} (ascertained from the positive value of the thrust equation) and substituted in the equation $P = H_1 + H_2$; the general equations for a horizontal load P applied at B are

$$\frac{4C_1}{hP} = \frac{r(7K_1 + 8) + 12h(K_1 + 1) + \frac{4h^2}{rK_2}(3K_1K_2 + 2K_1 + 2K_2 + 1)}{2r + 6h + \frac{h^2}{rK_2}\left(6K_2 + \frac{K_2}{K_1} + 1\right)} \quad . \quad (11)$$

$$\frac{4\beta_1 C_2}{hP} = rK_2 + 6h\left(1 + 2K_2 + \frac{K_2}{K_1}\right) + \frac{4h^2}{rK_1}(3K_1K_2 + 2K_1 + 2K_2 + 1) \quad . \quad (12)$$

where β_1 is the denominator on the right-hand side of (11).

$$\frac{2\beta_1}{hP}.M_{BA} = 4r + 9h + \frac{h^2}{rK_2}(6K_2 + 2) \quad . \quad . \quad . \quad (13)$$

$$\frac{2\beta_1}{hP}.M_{CD} = (6K_1 + 2)\frac{h^2}{rK_1} + 3h \quad . \quad . \quad . \quad . \quad (14)$$

H_1 and H_2 are the bending-moment terms divided by h.

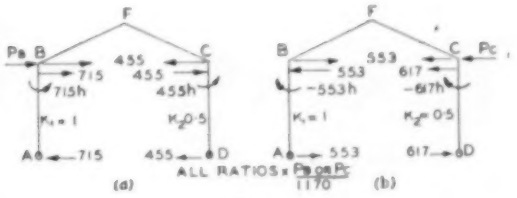


Fig. 7.

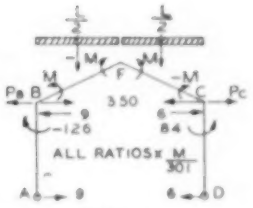


Fig. 8.

In the example in *Figs.* 7(*a*) and 7(*b*) the following are assumed: $h = 14$ ft.; $S_0 = 14$ ft.; $r\left(\frac{L}{4}\right) = 6$ ft.; $K_0 = 1$; $K_1 = 1$; $K_2 = 0{\cdot}5$.

For load P acting at B,

$$H_1 = \frac{715P}{1170};\ H_2 = \frac{455P}{1170};\ M_{BA} = \frac{715hP}{1170};\ M_{CD} = \frac{455Ph}{1170}.$$

$$C_1 = \frac{5590hP}{4680};\ C_2 = \frac{2795hP}{4680};\ \Delta_2 = \frac{C_2h}{6EK_2} = \frac{5590hP}{28{,}080E}.$$

For P acting at C,

$$H_1 = \frac{553P}{1170};\ H_2 = \frac{617P}{1170};\ M_{BA} = -\frac{553Ph}{1170};\ M_{CD} = -\frac{617hP}{1170};\ \Delta_1 = \frac{5590hP}{28{,}080E}.$$

For comparison with the fixed-base example a uniform load is assumed to act on the horizontal projection of the rafters (*Fig.* 8). By moment distribution the fixing moments at B and C are $M_{BF} = \frac{18M}{43} = -M_{BA}$ and $M_{CD} = \frac{12M}{43} = -M_{CF}$.

H_0 is the same as for fixed column-bases and $H_0 = \frac{-15M}{2r}$. Substituting in the thrust equation,

$$H = \frac{50M}{3};\ P_B = \frac{341M}{301};\ P_C = \frac{344M}{301}.$$

$$\frac{M_{BA}}{M} = \frac{-18}{43} - \frac{341}{301} \times \frac{715}{1170} \times h + \frac{344}{310} \times \frac{553h}{1170} = \frac{-497}{195}.$$

Therefore $H_1 = \frac{71M}{390} = -H_2$, and $\frac{M_C}{M} = \frac{497}{195}$.

Method applied to Frames with Imperfectly-fixed Bases.

If the foundation of a frame is such that neither the fully-fixed nor the perfect hinge conditions apply, the frame may be designed for a predetermined bending moment as follows. The thrusts and bending moments due to M_A (*Fig.* 9*a*) when the base at D is assumed to be hinged are calculated. The thrusts and bending moments due to M_D (*Fig.* 9*b*) when the base at A is assumed to be hinged are also calculated. These moments and thrusts are then subtracted or added to the moments and thrusts of the fixed-base frame. Any intermediate degree of fixity of the column bases may by this procedure be found.

TABLE III.

	A	6 8 7	7 C 3	D
	MA	$\frac{1}{2}$		
		$-\frac{5}{26}$ $-\frac{7}{26}$	$\frac{1}{26}$	
		ETC.	$\frac{-7}{260}$ $\frac{-3}{260}$	
SUM	MA	$\frac{23}{86}$ $\frac{-23}{86}$	$\frac{1}{86}$	$\frac{-1}{86}$

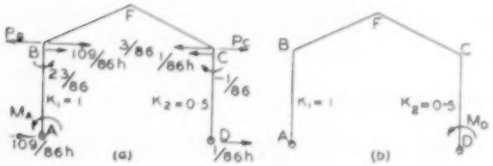


Fig. 9.

A frame of similar properties to that shown in *Fig.* 2, but hinged at D, is used to illustrate the application of this method. The moment M_A is distributed as shown in *Fig.* 3.

The stiffness ratios of column to rafter at B and C are as 6 is to 7 and as 3 is to 7 due to hinged conditions at A and D. Substituting in the thrust equation,

$$\frac{H_{ra}}{M_A} = \frac{3}{4r}\left(\frac{-23}{86} - \frac{1}{86}\right) = \frac{-3}{86}; \quad \frac{H_1}{M} = \frac{109}{86h}; \quad \frac{H_2}{M} = \frac{1}{86h}.$$

The thrusts and bending moments calculated in *Table* III are shown in *Fig.* 9(*a*). The balancing force P_B is $\frac{3}{86} + \frac{109}{86h} = \frac{151}{86h}$ and acts to the left at B; P_C is $\frac{42}{86h} + \frac{1}{86h} = \frac{43}{86h}$ and acts to the right at C. As the force P_B acts in the opposite direction to that in *Fig.* 7(*a*), and P_C in the opposite direction to that in *Fig.* 7(*b*),

$$\frac{H_1}{M_A} = \frac{109}{7 \times 172} - \frac{151}{86 \times 14} \times \frac{715}{1170} + \frac{43}{86 \times 14} \times \frac{553}{1170} = \frac{4}{130} = -\frac{H_2}{M_A}.$$

$$\frac{M_B}{M_A} = \frac{23}{86} - \frac{151 \times h}{86 \times 14} \times \frac{715}{1170} - \frac{43 \times 553h}{86 \times 14 \times 1170} = \frac{-74}{130}.$$

$$\frac{M_C}{M_A} = \frac{-1}{86} - \frac{151 \times h}{86 \times 14} \times \frac{455}{1170} + \frac{43 \times 617}{86 \times 14 \times 1170} = \frac{-56}{130}.$$

The distribution of M_D when column-base A is assumed to be hinged gives $H_1 = \frac{31}{780}$; $M_D = -H_2$; $M_B = \frac{-217}{390}$; $M_C = \frac{-173}{390}$. These values are shown in *Fig.* 10.

The bending moments and thrusts on a frame with hinged bases may be obtained from those on a frame with fixed bases by applying to each fixed base a bending moment equal and opposite to the fixed-end moment. To demonstrate

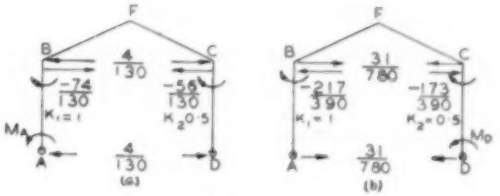


Fig. 10.

this application the bending moments for the frame shown in *Fig.* 8 are derived from the frame with fixed bases shown in *Fig.* 4, thus

$$M_B \text{ (hinged bases)} = \frac{-5866}{2215} - \frac{74 \times 3626}{130 \times 2215} + \frac{217}{390} \times \frac{4106}{2215} = \frac{-497}{195}.$$

$$M_C \text{ (hinged bases)} = \frac{5386}{2215} - \frac{56}{130} \times \frac{3626}{2215} + \frac{173}{390} \times \frac{4106}{2215} = \frac{497}{195}.$$

This method may also be applied to frames with one base fixed and one base hinged.

Broken Wires in a Prestressed Tank.

A WATER tank recently built at Richmond, California, is 200 ft. diameter by 44 ft. high and has a capacity of twelve million gallons. It contains 360 parallel prestressing wires 170 ft. long, each anchored in two pilasters, of which there are eight in the circumference. Each cable comprises eighteen high-tensile wires of 0·196 in. diameter enclosed in a metal tube, and these were fixed in the shutters before the concrete was placed. The wires were tensioned when the concrete had sufficient strength, but, as the losses due to friction were higher than were anticipated, the cables were not grouted because it was desired to re-tension the wires later. The re-tensioning was delayed for five months, and it was then found that some of the wires were corroded and broken. It was decided to renew the cables, and this work was started eleven months after the first cables were tensioned. It was then found that in most of the cables there were some broken wires, and that many of them were broken in more than one place. Details of the design of this tank were given in this journal for December 1954.

As a result of this failure an investigation was made of the prestressing wires in the ring beam of another tank now being built in the same district. A hole was cut in the concrete and the wires were found to be in good condition. The thin grout had, however, segregated, and at one place the upper part of the duct was full of water. The foregoing is abstracted from "Engineering News-Record" for June 2, 1955.

The Correlation between Calculated and Observed Stresses and Displacements.

A CONFERENCE on the "Correlation between Calculated and Observed Stresses and Displacements in Structures" is to be held by the Institution of Civil Engineers in London on September 21 and 22, at which twenty papers will be presented and discussed. There will be sessions on "The Philosophy and Techniques of Testing", "Steel Structures", "Concrete Structures", and "Earth Pressures and Movements". All engineers are invited to attend and take part in the discussions. Further information is obtainable from the Secretary of the Institution, Great George Street, London, S.W.1.

Book Reviews.

"Building Materials: Their Elasticity and Inelasticity." Edited by M. Reiner. (Amsterdam: North Holland Publishing Co. 1954. Price £4 4s.)

THE study of the flow and deformation of matter is a subject of considerable importance to those interested in the behaviour of plain and reinforced concrete under various conditions of stress. In this book, which is believed to be the first entirely devoted to the subject, introductory chapters on theoretical rheology by Dr. Reiner, and on its structural engineering aspects by Dr. Freudenthal, are followed by chapters dealing separately with various building materials. Concrete is dealt with by Mr. E. Forslind, who writes about the behaviour of fresh concrete mixtures, and by Dr. E. Torroja and Sr. A. Paez who are concerned with hardened plain and reinforced concrete. Materials considered by other authors include metals, wood, asphalt for roads, plasters, soils, and clay products. Although many readers will find these new conceptions of the non-elastic deformations of materials difficult to follow, they are of increasing importance with the complexity of modern structures and the book deserves serious study.

"Steel Designers' Manual". By C. S. Gray, L. E. Kent, W. A. Mitchell, and G. B. Godfrey. (London: Crosby Lockwood & Son, Ltd. 1955. Price £2 10s.)

THIS comprehensive book contains all the information necessary for the design of steel-framed buildings and, in addition, the first half, of about 400 pages, is a treatise on the analysis of statically-indeterminate structures. The methods described (the moment-area theorems, moment distribution, column analogy, and slope-deflections) are, of course, applicable to all elastic structures and are those found most useful in design offices. Tables and diagrams giving shearing forces, bending moments, and deflections of freely-supported beams and beams with fixed ends for many arrangements of the loads as well as formulæ for the direct calculation of the bending moments on various shapes of rigid frames are features of the book: these latter will be useful to those who are not acquainted with the works of Professor Kleinlogel. Emphasis throughout is on the application of the methods to practical problems, and there are many completely-worked examples illustrated by clear diagrams. The treatment of prismatic members is exhaustive but, although the comparative neglect is understandable in a book dealing primarily with steelwork, a little more attention might have been given to members of varying moment of inertia. The remainder of the book includes chapters dealing with bunkers, the torsional strength of steel beams, welding, design of steel members subjected to bending and direct forces, composite members of steel and concrete, "plastic" design, foundations, glazing, sheeting and flooring details, and a complete design for a double-bay, single-story workshop. Little information in addition to that contained in this book will be found necessary in day-to-day work in the design of rigid steel frames. It is an excellent book.—J. E. G.

"Beton-Kalender, 1955." Parts I and II. (Berlin: Wilhelm Ernst & Sohn. Price 16 D.M.)

CHANGES in this, the 44th, edition include a chapter on the design of prismatic structures, the enlargement of the section on prestressed concrete, and the rearrangement of the chapter on floors in order to show more clearly the properties of different types of construction. This work contains formulæ, tables, and diagrams that can be readily used by a designer with little or no knowledge of the German language, but those with a knowledge of the language will find much helpful information on a wide range of problems.

"As Linhas de Influência nos Pórticos Continuous Planos e do Espaço Tridimensional." By Samuel Chamecki. (Rio de Janeiro: Editora Cientifica. No price stated. Printed in the Spanish language.)

DESCRIBES a method of designing three-dimensional continuous frames by a "displacements propagation process" of compressing the displacements at the joints. It is claimed that the method is a quick one, and gives a clear view of the effects of forces in a frame.

Prestressed Bridges in Belgium.

RESTRAINED BEAMS WITH ADJUSTABLE SUPPORTS.

THE construction of the third section of the Nimy-Blaton canal in Belgium has necessitated building eight road-bridges of spans varying between 150 ft. and 190 ft. An elevation of one of the bridges is shown in *Fig.* 1. All the bridges, which are described by M. J. Capel in "Travaux" for December, 1954, are of the same type and comprise five to eight beams of varying depth; the differences in the spans are made by varying the length of the central section which has a constant depth of 4 ft. 11 in. The beams are T-shaped with a top flange about 5 ft. wide and are placed at about 6-ft. 6-in. centres, the spaces between the beams being filled with concrete after the beams are cast. Although 53 beams of the same cross section are required the contractor preferred, because of site conditions, to cast the beams in place rather than to precast them.

The supports at the ends of the superstructure of the bridge comprise four inclined rockers which are supported on foundations inclined at the same angle as the interior rockers (*Fig.* 2). The superstructure may be considered as a continuous beam supported at four points, the short end spans being tied by cables to the foundations; thus the bending moments may be adjusted by flat jacks at the interior supports. The forces, due to the dead and live loads, acting along the axis of the interior rockers have a horizontal component which reduces the prestressing force required in the deck. This horizontal force is increased by a rise in temperature and produces bending moments which reduce the deformations brought about by rise in temperature. The resultant of the forces due to the loads on the superstructure passes through the intersection of the axes of the rockers. The displacement of this point is negligible compared with the rotation of the rockers produced by variations in the load; this point acts, therefore, as a pin.

Fig. 1.—Elevation.

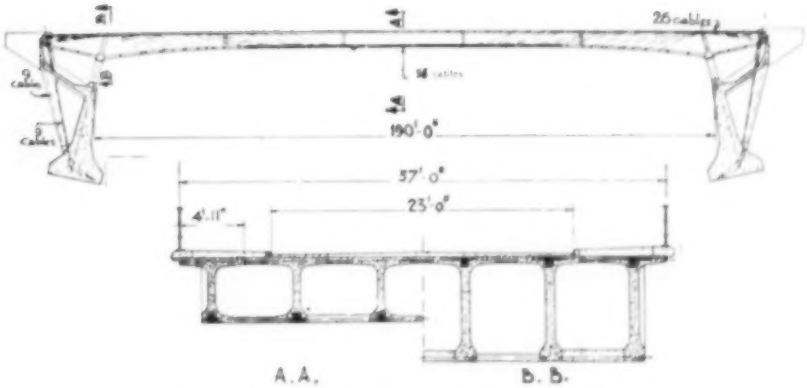


Fig. 2.—Longitudinal and Cross Sections.

Thus it is advantageous to incline the interior rockers towards the centre of the span and to curve upwards the intrados of the superstructure so that the line of pressure, at least for the dead loads, approaches as nearly as possible the centre-line of the superstructure.

The flat jacks may be adjusted at any time to compensate for small settlements of the foundations. The inclination of the foundation is at an angle of less than 15 deg. in order to ensure that the horizontal component of the force is small. The ground pressure does not exceed about 3¾ tons per square foot.

Figs. 1 and 2 show one of the bridges which has a span of 190 ft., a width between the railings of 37 ft., and a carriageway of 23 ft. The depth of the beams at midspan is 4 ft. 11 in., the webs are 6·3 in. wide, the bottom flange 1 ft. 7 in. wide and the top flange 4 ft. 11 in. At the interior supports and towards the top of each beam there are 26 cables of which 16 curve downwards and are continuous across the span. The remaining ten cables are anchored in the bottom flange of the beam. The other ends of all the cables are anchored in the ends of the beam over the external support. The outer ends of the beams are held down by nine cables passing through the outer rockers and struts to the foundations. The lower flanges of the beams are connected at intervals of about 33 ft. by struts which are prestressed, and cables are also placed transversely at the same intervals through the top flanges of the beams. Each cable contains twelve wires of 0·2 in. diameter.

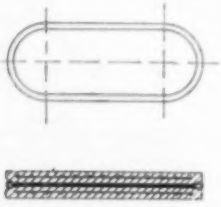


Fig. 3.—A Flat Jack.

The ends of the beams, in which the anchors for the cones are placed, were precast because of the large number of anchors in the comparatively small member (*Fig.* 4).

The outer rockers, through which the cables pass to the foundations, at the ends of the beams are shown at *a* in *Fig.* 5 and in detail in *Figs.* 6 and 7. They are 3 ft. 6 in. thick and lightly reinforced with mild steel bars. The interior rockers (*b* in *Fig.* 5) are 1 ft. 8 in. by 2 ft. 3 in. The upper and lower parts of these rockers were precast and the reinforcement is arranged to follow the lines of principal stress (*Fig.* 8). Tests of similar members at an age of 37 days showed that the first cracks occurred at a load of 430 tons and failure at loads exceeding 870 tons, compared with a working load of 325 tons. The lower ends of these members are seated on the flat-jack assemblies comprising oblong jacks encased in reinforced concrete.

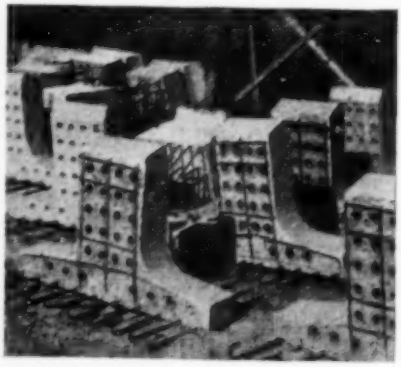


Fig. 4.—Precast Ends of Main Beams.

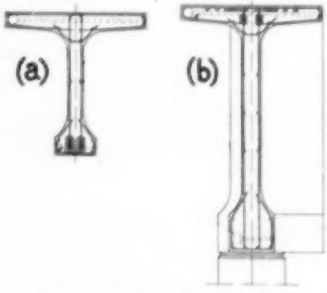


Fig. 5.—Cross Sections.

The flat jacks are composed of two parallel flat steel plates with a small space between them. These plates are joined at their edges by another steel plate bent

in the shape of a torus (*Fig.* 3). A steel pipe fixed to one of the plates allows the space between the plates to be filled with a liquid under pressure which separates the plates and thus allows the level of the support to be adjusted.

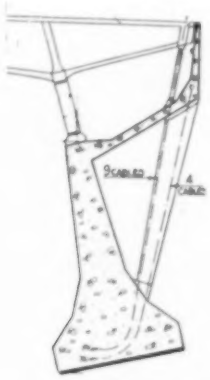


Fig. 6.—Arrangement of Rockers.

The stresses for which the bridges are designed are: Concrete in compression, 2000 lb. per square inch during construction and 1775 lb. per square inch under working loads; a tensile stress in the concrete of 142 lb. per square inch

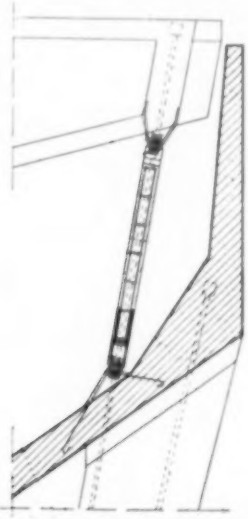


Fig. 7.—An Outer Rocker.

was allowed during construction but no tensile stresses were to occur under working loads. The initial stress in the steel was 142,000 lb. per square inch, and the

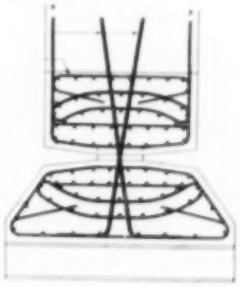


Fig. 8.—Reinforcement of Ends of Internal Rockers.

stress, after all losses, 121,000 lb. per square inch. The average compressive strength of 8-in. cubes was 4675 lb. per square inch at seven days and 6780 lb. per square inch at 18 days.

Construction.

The beams were cast in-situ in metal-lined timber shutters, the concrete being consolidated by immersion vibrators. At the beginning of the work one beam only was cast each week, but later it was found possible to cast a beam every four days.

The order of prestressing was as follows.

(1) Three days after placing the concrete sufficient of the longitudinal cables were tensioned to enable a beam to carry its own weight as a freely-supported beam on the two internal rockers.

(2) Transverse prestressing between all the beams forming a bridge.

(3) Tensioning of the cables passing through the external rockers to the foundations.

(4) Completion of the longitudinal prestressing.

(5) Grouting of the cable ducts.

(6) Inflation of the flat jacks. These were initially inflated with water and the pressure maintained for a week, when it was considered that most of the creep of the concrete had taken place, and cement grout was then injected into the jacks.

The quantities of materials required per square foot of the bridge deck were

2 cu. ft. of concrete, 7¼ lb. of high-tensile steel, and 3¾ lb. of mild steel.

Work on the first bridge was started in June, 1951, and the last was completed in 1954. The work was carried out for the Administration des Ponts et Chaussées. The design was by the Société Technique pour l'Utilization de la Précontrainte, and the contractors were Société des Entreprises Socol.

Cantilevered Roof in Stockholm.

THE roof shown in *Fig.* 1 is over the entrance to an underground station in Stockholm. It consists of a doubly-curved "shell" supported by beams cantilevering from two central columns with a transverse beam connecting the columns and cantilevering to support the parts of the roof which project in a transverse direction beyond the columns.

The length of the roof is 57 ft.; its greatest width is 46 ft. 9 in., reducing to 24 ft. 8 in. at the ends. The central part is 3·15 in. thick and the parts projecting at the sides are 4·7 in. thick at the longitudinal beams reducing to 2·4 in. at the edges. A plan and sections (with dimensions in the metric system) are given in *Fig.* 2. The foregoing is abstracted from a recent number of "Beton- und Stahlbetonbau".

Fig. 1.

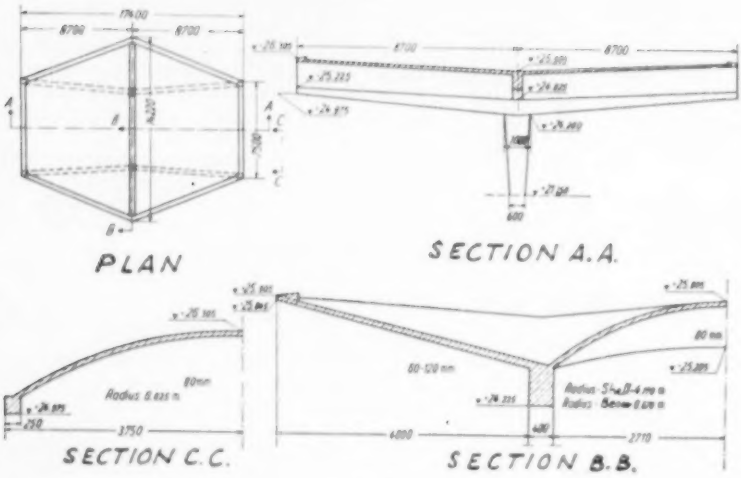


Fig. 2.

The Construction of a Canal Lock in Holland.

In a recent number of the Dutch journal "Cement" the construction of the lock at Tiel on the Amsterdam-Rhine canal is described by Mr. B. A. Gelders. The lock with the gate-portal is shown in *Fig.* 1. Work was started in the year 1940 on the inner-gate abutments at the canal end of the lock, together with the approach channel on the landward side of the river dyke and some earthworks at the approach to the north end of the lock. The work began with the excavation of a foundation pit with the cross section shown in *Fig.* 2. The earth was removed in two layers by a dredger-type excavator and a drag-line. Part of the

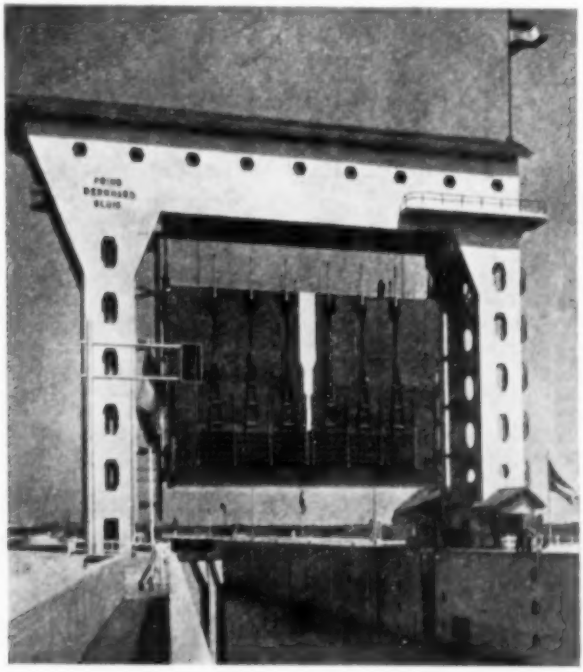


Fig. 1.—The New Lock at Tiel.

foundations of railway and road bridges, and a system of dolphins with fenders at the north entrance of the lock, but was stopped in 1942. These works were resumed at the end of 1946 and completed in March, 1948. The contractors were N. V. Aanneming Mij. v/h Hillen en Roosen, Amsterdam.

Excavation and Drainage.

In May, 1948, tenders were invited for the completion of the lock, which is about 1300 ft. long by 60 ft. wide by 44 ft. deep, including excavation of part of the spoil was used for the construction of the dykes along the approach channel from the river, the remainder being used as backfill for the foundation pit. The excavation was drained by well-points of both Dutch and British manufacture. In both types the centrifugal pump with its electric motor was placed at the bottom of the well-shaft, the lower ends of the filter tubes being about 65 ft. below the datum level. Eleven well-points at intervals of about 145 ft. were sunk from the original ground-level along the edge of the future east slope to enable the

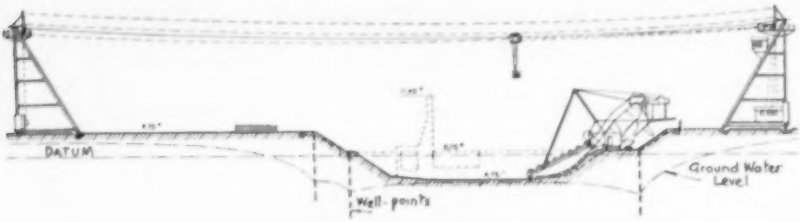


Fig. 2.—Excavation of Lock.

excavation to be carried out for the intermediate berm. Fifteen well-points were then sunk from the level of this berm and five more from ground-level along the south side of the excavation. In this way it was possible to remove the top layer of earth down to the level of the berm. After twenty-two more well-points had been placed along the west berm and three at the south-west corner it was possible to excavate the second layer. The water was discharged from the pumps through pipe-lines into a ditch along the west boundary of the site. An auxiliary pumping station at the foot of the inner slope of the main river dyke pumped the water into the approach-channel to the river. At the highest water level in the river Waal the amount of water to be discharged would be nearly 450 gals. per second and the pumping station was equipped with two pumps each with a capacity of 225 gals. per second and three pumps each with a capacity of 67 gals. per second thereby ensuring sufficient capacity in reserve. The average discharge was about 225 gals. per second. The effect of the dewatering on the level of the ground-water was noticeable to a distance of over one mile on each side of the excavation. After the diversion of part of the railway line between Tiel and Elst, the pit with its dewatering system was extended northwards.

Pile Casting and Driving.

Concrete piles were made on the site on a concrete bed extending along the whole length of the excavation. The piles are 13¾ in. square and chamfered along the edges of the heads to reduce the risk of damage during driving. The shutters for the sides of the piles, with

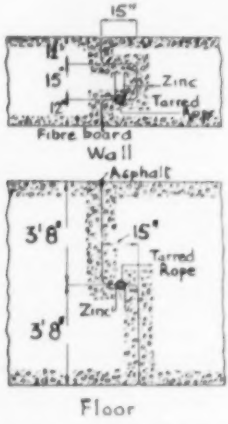


Fig. 3.—Joints in Walls and Floor of Lock.

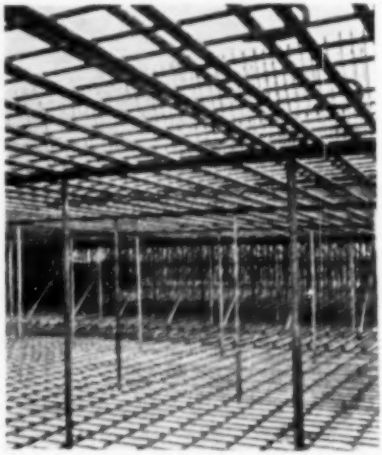


Fig. 4.—Supports for the Top Reinforcement in the Base Slab.

intermediate spaces equal to the width of one pile, were placed on oiled paper on the bed, and on removal of the shutters the spaces between the piles were used as moulds. The sides of the piles already cast were coated with clay slurry to prevent adhesion. Each layer of piles was covered with a layer of paper; a second layer of piles was then cast on the first layer, followed by a third and a fourth. The reinforcement in the piles comprises four $\frac{3}{4}$-in. bars and four $\frac{5}{8}$-in. bars with $\frac{1}{4}$-in. helical binding. The concrete contained 670 lb. of cement per cu. yd. and was compacted by internal vibrators. The concrete was transported by an aerial ropeway arranged in a direction at right-angles to the centre-line of the lock.

The piles were driven by three "Dutch" pile-frames using semi-automatic steam-hammers of 4 tons weight. Each hammer drove an average of fifteen piles a day. The piles were jetted into the ground to within about 3 ft. of their final penetration. It was not necessary to strip the heads of the piles as the lock structure rests on the pile heads and is thus able to undergo elastic deformations without being restrained by the piles. Where necessary, the pile-heads were patched and rendered, and all of them were given a coat of tar. The pile-heads projected about 1 in. through the "blinding" layer of concrete.

As the blinding was laid short ribs, at an angle of 45 deg. to the centre-line of the lock, were cast. These ribs are 2 in. thick and maintain the reinforcement in position.

Welding Reinforcement.

The reinforcement used in the completion of the lock consisted of $\frac{1}{2}$-in. to 2-in. bars. The bars greater than $\frac{1}{2}$ in. were for the most part supplied in short lengths and, to avoid numerous laps, a resistance-welding machine for making butt welds was used to weld bars of one diameter, the required lengths then being cut by an electrically-operated cutting machine placed behind the welding plant.

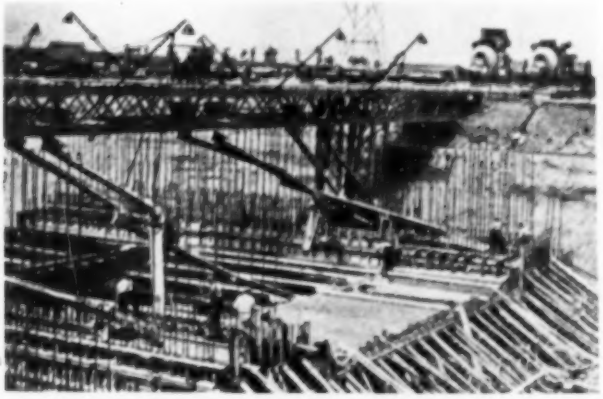


Fig. 5.—Concreting the Base Slab.

Before welding, the bars were cleaned with an emery wheel at their ends and at the electrode clamps. A saving of about 200 tons of reinforcement in the original requirement of about 6000 tons was made by welding.

The cut and bent reinforcement was conveyed to the foundation pit by the aerial ropeway with the aid of a spreader beam.

Concreting the Floor and Walls.

After the driving of the bearing piles and the sheet-pile walls the construction of the gate abutments and the lock-chamber was begun. Because of the joints in the floor the parts with the lower members of these joints were concreted first. *Fig.* 3 shows how the joints were sealed by tarred rope soaked in asphalt. After the end faces of these sections had

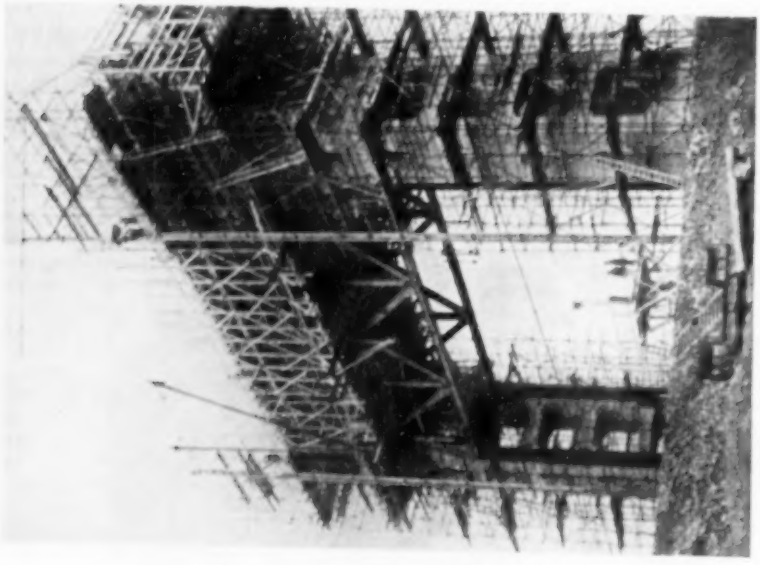


Fig. 7.—Shutters for the Towers of the Lock Gate.

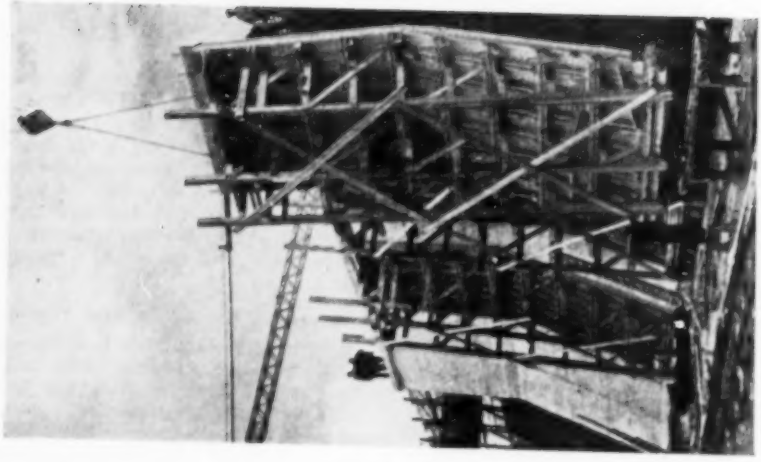


Fig. 6.—Moving the Outer Shutters of the Walls.

been coated with tar the intermediate sections were concreted. The reinforcement in the top of the floor was supported by steel angles (*Fig.* 4).

A travelling gantry, with two narrow-gauge railway tracks at ground level, was used for concreting the floors (*Fig.* 5). Two concrete mixers, each with a capacity of about 1½ cu. yd., were placed at one end of the gantry. The concrete was conveyed in wagons with bottom-opening doors to one of the six hoppers attached to the gantry and placed by swivelling chutes capable of reaching every part of the section of the floor under construction. The volume of concrete in the sections of the floor varied from 830 to 1620 cu. yd., and each section was placed in one continuous operation. The same installation was used for placing the concrete for the second and third lifts of the culverts and walls. The capacity of the plant was about 34 cu. yd. per hour.

Shuttering.

Travelling shutters were used for the inner faces of the walls of the lock-chamber. They were constructed of tubular steel scaffolding and were provided with two rows of wheels, each row having three wheels (*Fig.* 8). The faces of the shutters were plumbed by the use of three jacks.

In placing the concrete for the fourth lift, two sections of wall opposite each other were concreted simultaneously, using the aerial-ropeway to transport the concrete. The shutters for each of these sections was, after adjustment, connected together at the top by steel beams. For the outer faces of the walls, splayed shutters, about 13 ft. high by 33 ft. long, were used, and these were connected by tie-rods to the travelling shutters. To strike the shutters the ends of the tie-rods were unscrewed and the jacks under the travelling shutters released, so that the travelling shutters came away from the wall and could be moved to the next section. The shutters for the back of the wall were moved by the aerial ropeway (*Fig.* 6).

The reinforcement in the wall was fixed to steel frames spaced at about 11 ft.

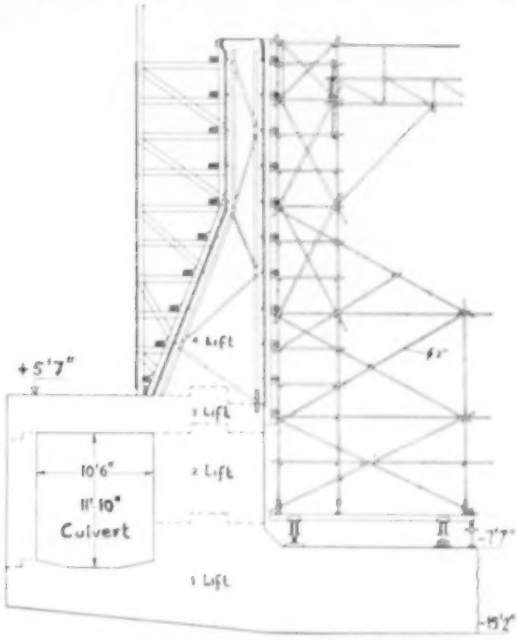


Fig. 8.—Shutters for the Walls.

intervals. These frames were welded to the bars projecting from the third lift and to some of the longitudinal bars, resulting in a high degree of rigidity.

The shape of the gate-abutments and of the housings for the sluice-gates of the culverts did not allow travelling or standardised shutters to be used. In constructing the two towers, concreting was carried out in lifts of about 12 ft. until the floor level of the chamber for the gate-operating machinery was reached. To support the shutters and wet concrete for this chamber two steel trusses were used. These had a span of about 76 ft. and rested on stanchions consisting of broad-flange beams fixed to the towers. The two walls were first concreted to roof level in one continuous operation, then the floor was placed, and finally the roof and the gutters (*Fig.* 7).

Construction of the Dolphins.

Dolphins consisting of reinforced concrete shafts and fenders were constructed along the centre-line of the approach channel from the river. Some of the dolphins are connected to one another by prestressed foot-bridges with a span of about 66 ft. The shafts were cast in situ in four sections. The lowest portion, which was provided with a cutting-edge with a thicker wall immediately above it, was cast in a small excavation the bottom of which was a little below the lowest water level expected in the approach channel to the river. After the concrete had hardened and the shutters were removed, this portion was sunk by digging away the earth under it by means of a small grab suspended from a crane. This was continued until the top of the shaft was level with the bottom of the excavation. In carrying out this work it was necessary to ensure that the difference in level of the water inside and outside the shaft did not become too great. The second portion was then concreted, and the sinking was continued until the lower edge reached the required depth, about 43 ft. below the original ground level. Concrete was then placed through a vertical pipe to form a seal at the bottom of the shaft, the remaining part of which was filled with sand. The position of the top of the second lift was then checked. The greatest deviation from the vertical was found to be about 1 ft. and in most cases was much less. A transition section about 1 ft. 6 in. high was used to bring the shaft to the correct line and the remainder of the shaft was then concreted in two lifts, this part of the shaft also being filled with sand. Lengths of gas-pipe were embedded in the walls of the shaft a little above the level of the bottom of the future approach channel to provide drainage for the sand, a layer of gravel replacing the sand at this level. The foot-bridges resting on the dolphins were concreted from scaffolding supported from the original ground level as the approach channel had not been excavated when these were built.

Concrete.

The concrete used for the construction of the lock contained 575 lb. of cement per cubic yard. The concrete for the lifting-gate towers and the machinery chamber contained 605 lb. of cement per cubic yard, and that for the prestressed foot-bridges contained 675 lb. per cubic yard. The concrete for the floor and walls of the lock had a slump of 5½ in. and was compacted by hand. The concrete in the towers, the portal, and the foot-bridges was compacted with internal vibrators. The walls of the lock were rendered with cement slurry. The walls of the towers, the portal, and the small control building were rubbed down with rags soaked in cement mortar composed of 1 part of cement to 2 parts of silver sand.

The contractors for the second portion of the work were Nederlandse Aanneming Mij. v/h fa. H. F. Boersma and Aannemersbedrijf Dirk Verstoep, of The Hague.

An Inspection of Roads on the Continent.

A VISIT to Denmark, Germany, and Holland to inspect methods of constructing concrete roads has been arranged by the Cement and Concrete Association. The tour will take place from August 21 to August 28, and further details may be had from the Pavings Development Group of the Cement and Concrete Association, 52 Grosvenor Gardens, London, S.W.1.

The Flow of Materials from Bins.

In a paper on the flow of materials from bins, Mr. John K. Rudd (of the Richardson Scale Co. of the U.S.A.) states that the most important factors affecting the flow are the diameter of the hopper, the slope of the sides and the diameter of the discharge opening, the coefficient of friction, the ratio of lateral pressure, and the density, head, and shearing strength of the material. Temperature and moisture content may be indirectly accounted for or not allowed for if the sample tested is representative of the material. To start the flow only the forces acting on a central column of material extending up from the discharge opening have to be considered. Material flowing from a confined area should be classified as either cohesive or non-cohesive. The flow of the two classes is so different that it is best to consider them as two different problems. There is a minimum discharge opening for any given cohesive material in any given hopper or bin.

Three investigations have been made, including (1) Measurements of the pressure created by a head of material in hoppers with vertical and sloping sides. These tests were made primarily to check Janssen's equation for pressures in circular straight-sided hoppers, and to see how far the equation applied to hoppers with sloping sides. (2) Investigation of lateral and vertical pressures created by materials in bins of different shape. Lateral and vertical pressures, under a given head of liquid, are always equal, but this is not true for solids because of the friction between particles and cohesion. (3) Investigation of the mechanics of flow.

Janssen and Airy both produced equations for the pressure exerted by material in straight-sided bins and hoppers. Janssen's equation is

$$P_v = \frac{wR}{K\mu'}\left(1 - e^{-\frac{K\mu' h}{R}}\right),$$

where P_v is the pressure on the base in lb. per square foot, R is the hydraulic radius (ft.), K the ratio of lateral and vertical pressures in the bin, μ' the coefficient of friction of the material against the wall of the bin, h the height of material in the bin above the base (ft.), and w the specific weight of material (lb. per cubic foot). Airy's equation is

$$P_v = \frac{wD}{K(\mu + \mu')} \times \left(1 - \frac{\sqrt{1 + \mu^2}}{\sqrt{\frac{2h}{D}(\mu + \mu') + 1 - \mu.\mu'}}\right),$$

where D is the diameter of the discharge, μ' the coefficient of friction of the material against the wall, and μ the coefficient of friction of material against material. Janssen's equation was chosen because it seemed sounder, and also it is simpler.

To check Janssen's equation, a small straight-sided hopper and hoppers having slopes of 5 deg. and 20 deg. from the vertical were built. An hydraulic cell was placed under the hopper at the discharge opening. Tests were made using $\frac{1}{4}$ in. polystyrene granules, Portland cement, and dry mash feed. Pressures on the full discharge opening were determined for known heads of material in the hopper. The results of the tests with the circular straight-sided hopper agreed with theoretical values obtained from Janssen's equation, but they also indicated that this equation did not apply to hoppers with sloping sides.

After preliminary tests, it was thought that a more general equation than Janssen's could be written and which would apply to hoppers with sloping as well as straight sides. K (the ratio of lateral to vertical pressure of the confined solid) has a theoretical basis, as shown by Rankine, of $\frac{1 - \sin\delta}{1 + \sin\delta}$, where δ is the angle of repose. However, this applies only to materials in an unconfined condition. So far only a few values of K for confined materials have been published, and those by Janssen for dry grains. To get values of K a tank was made into which was placed a cylinder with walls of extremely thin flexible rubber and with rigid ends. A sample of material was placed in the cylinder and the cylinder installed in the tank. A pressure was established inside the tank to produce a known lateral pressure on the sample. A vertical pressure was then applied by a piston on top of the cylindrical sample, until the rubber walls

started to give way. At this point the vertical and lateral components $\frac{P_h}{P_v} = K$ were determined.

It was found that K varied with the height of the material in the hopper and the diameter of the hopper. Therefore, in determining K, the height-to-diameter ratio of the cylinder was chosen to comply with the shape of the hopper. With the values of K determined, it is possible to check test results with theoretical figures, and to draw test and theoretical curves (*Fig.* 1). From these data it is believed that a general equation can be evolved.

To study the flow inside bins, a bin was built with a glass front, a metal back, an adjustable discharge opening, and tapering lower sides. Cohesive and non-cohesive materials, and bins of several shapes, were used, and slow-motion pictures were taken of the flow of material. These tests indicate that (1) A central column of flow exists above the discharge opening and is as wide as the opening. If material is being put into the bin at the same rate as it is being withdrawn, material outside the area of the central column remains static. (2) Material flows into the area of the central column

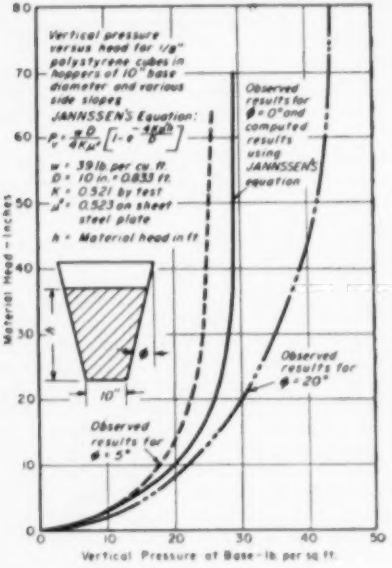

Vertical pressure versus head for 1/8" polystyrene cubes in hoppers of 10" base diameter and various side slopes
JANNSSEN'S Equation:
w = 39 lb. per cu. ft.
D = 10 in. = 0.833 ft.
K = 0.521 by test
μ' = 0.523 on sheet steel plate
h = Material head in ft.
Observed results for φ = 0° and computed results using JANNSSEN'S equation
Observed results for φ = 20°
Observed results for φ = 5°
10"
Material Head - Inches
Vertical Pressure at Base - lb. per sq. ft.
0
10
20
30
40
50
60
70
80

Fig. 1.

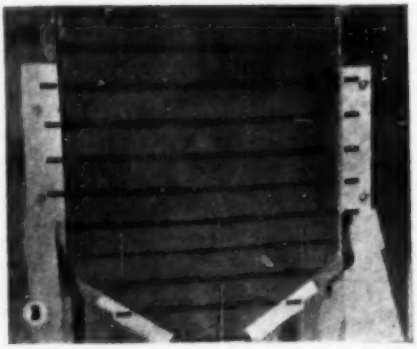
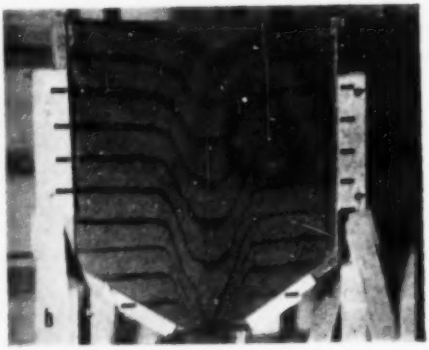


Fig. 2.



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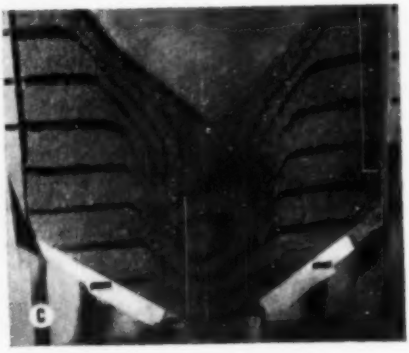

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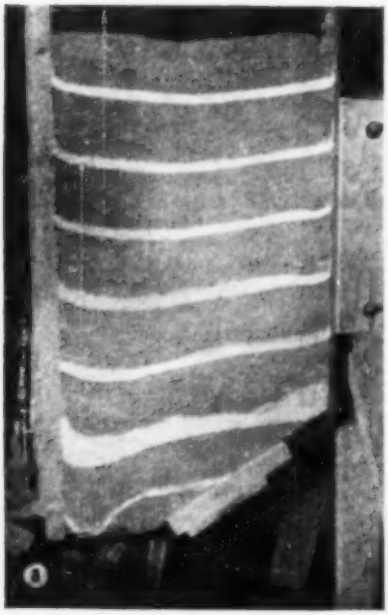

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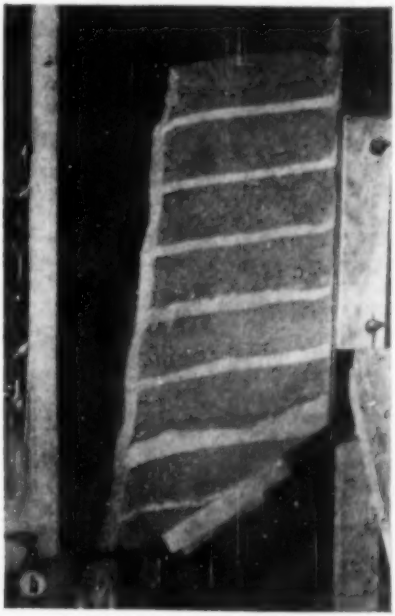

Fig. 6.

as soon as the central column is discharged (*Figs.* 2 to 4). (3) With cohesive materials a central " plug " moves out when the discharge gate is opened, and may leave a hole (*Figs.* 5 and 6). To get further flow in this case requires rapping or hammering on the bin.

It is believed that flow occurs when a stress is set up that overcomes the shearing strength of the material. It is possible to find the shearing strength of a material in any shape of bin by measuring the force required to make a known plug of material separately shear from surrounding material. The amount of stress required depends on the characteristics of the material and the shape of the bin. When stress and shearing strength are both known, conditions governing flow can be predicted. To confirm theoretical findings, it is proposed to build experimental hoppers, one with a discharge opening that will permit flow, and another with a slightly smaller opening that will not allow flow. [The foregoing is from " Rock Products " for March 1954.]

Courses in Concrete Practice.

THE course in concrete practice established by the City and Guilds of London Institute last year has been extended into a second year for the benefit of those students who wish to gain a more advanced knowledge of the subject. The second-year course will comprise one evening lecture per week for a period of 24 weeks, and will start with the session 1955/56. An examination, consisting of a written paper and an oral test, will be held in May 1956 and certificates (Grade II) will be awarded to successful candidates. Students will normally be expected to have completed successfully the first-year course. Full particulars can be obtained (price 6*d.*) from the City and Guilds of London Institute, Department of Technology, Gresham College, Basinghall Street, London, E.C.2.

Research on Roads.

THE following notes are abstracted from "Road Research 1954," the report of the Road Research Board for the year 1954 (H.M. Stationery Office. Price 4s.).

Skidding on Coarse Surfaces.

Most of the roads investigated because they had a bad record of skidding accidents had a rough coarse-textured surface, but the stones in the surface had become highly polished and rounded by the action of traffic. Of a total of 411 accidents occurring when the road was wet, 283 involved skidding.

Moisture under Roads.

When soil is covered with an impervious road the moisture content of the soil changes little with the seasons or with time. It has been found, however, that moisture penetrates under the road for distances up to 4 ft. from the verges.

Compaction of Soils.

Investigations have been made on the compaction of earthworks by the use of 4-cwt. and 2½-tons vibrating rollers, 4-cwt., 1½-tons and 2-tons vibrating-plate compactors, and 2-cwt. power rammers. The 4-cwt. vibrating roller gave reasonably satisfactory results with granular soils, but the depth of satisfactorily compacted soil was only about 2½ in.; very poor compaction was obtained with cohesive soils. The 2½-tons vibrating roller on granular soils had an effect equivalent to that of an ordinary 8-tons smooth-wheeled roller; on heavy clay vibration had little effect on compaction. Vibrating-plate compactors were not suitable for compacting heavy clays. The results obtained on granular soils with 2-tons and 1½-tons vibrating-plate compactors were similar; good compaction was produced with optimum moisture contents close to the average natural moisture contents of the soils in the British Isles. Rather lower compaction and corresponding higher optimum moisture contents were obtained with the 4-cwt. compactor. The heavier machines were capable of satisfactorily compacting loose layers 12 in. to 18 in. thick, and the 4-cwt. compactor loose layers 6 in. to 9 in. thick. Very similar performances were obtained from the 2-cwt. power rammers. Satisfactory compaction was produced on both granular and cohesive soils provided that the loose layers did not exceed 9 in. in thickness.

Settlement of Embankments.

Since May 1951 an investigation has been in progress on the approach embankments of a road bridge at Lackford, West Suffolk, to study the settlement. The underlying soils were organic clay and peat, both of which were highly compressible, and the embankments were about 4 ft. high. The settlement during the construction period of one year was up to 8 in., and in the following two years when the bridge was in use there was a further settlement of 2 in.

Reinforcement.

An experimental road has been built to determine whether a given weight of steel in a single layer is as effective in controlling cracking as the same weight in two layers. About 10 lb. of steel per square yard have been used (1) in a single layer of oblong-mesh near the top, and (2) near the bottom, (3) in two layers of oblong mesh of equal weight, one near the top and one near the bottom, (4) as in (3) but with both oblong-mesh and square-mesh fabrics arranged according to the Ministry of Transport and Civil Aviation specification for road and bridge works. The road was opened to traffic in April 1954 and no defects have yet occurred.

Prestressed Slabs.

Observations have been made on prestressed concrete roads at Ringwood and Crawley. Measurements of the movement of the slabs during daily temperature cycles indicated that the subgrade friction was high, the estimated coefficient of friction being near unity. Cracks in the slabs at Ringwood, and possibly those at Crawley, open during low temperature at night. This suggests that the stresses due to thermal movement exceed the prestress; at Ringwood, the designed prestress was not achieved because of difficulties arising from the methods used to form cable ducts. At Basildon it appears that some of the prestress is still maintained because cracks remain closed during low temperature.

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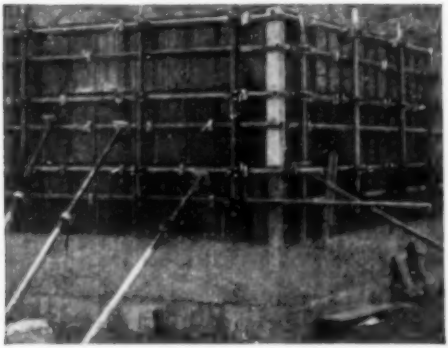

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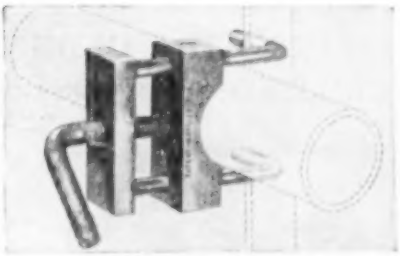

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Precast Frames Spanning 85 ft.

A FACTORY covering 40,000 sq. ft. now being built at Amman, Dumfriesshire, for the Airscrew Company & Jicwood, Ltd., for the manufacture of chipboard, is constructed almost entirely with precast concrete components. The main building (*Fig.* 1) was originally designed to comprise four bays of precast reinforced concrete frames of 43-ft. span at 15-ft. centres. However, it was decided that columns would be an embarrassment in this building and the frames were redesigned to span the full 85 ft. As many purlins had already been made it was decided to use the frames at 15 ft. centres, although normally for the wider span the frames would be spaced farther apart.

The factory consists of one section 85 ft. wide and 300 ft. long, an adjoining section of two bays 42 ft. 6 in. wide by 165 ft. long, and a boiler house 85 ft. wide by 30 ft. long. In the sections with the 85-ft. span the height to the eaves is 22 ft. 6 in., and in the two-span section 15 ft.

Except for the boiler house the roofs of the frames are pitched at 22½ deg. The frames spanning 85 ft. are made in four parts, namely two members each comprising a column and part of a rafter and two rafters. The column members are

Fig. 2.—Lifting a Column Member.

Fig. 1.—Building 300 ft. long by 85 ft. wide.

cast in batches of six one on top of the other (*Fig.* 5). The rafters are factory made and transported to the site. The bases for the columns are cast with sockets to receive the columns, and are tied to the concrete floor to resist the outward thrust. The columns are placed in the sockets, plumbed, and kept upright with scaffolding supporting shuttering projecting from the arm (*Fig.* 3). Precast rafters are then placed in position and the frames made monolithic with cast-in-situ joints at the apex and at the points of contraflexure. It was decided not to scarf the joints at the points of contraflexure as it was considered that this would require undue accuracy in the erection of heavy units and that such inaccuracy would cause high secondary stresses at the joints when the nuts were tightened. Each stanchion member weighs $4\frac{3}{4}$ tons; *Fig.* 2 shows the method of lifting them, and details of the joints are given in *Fig.* 4.

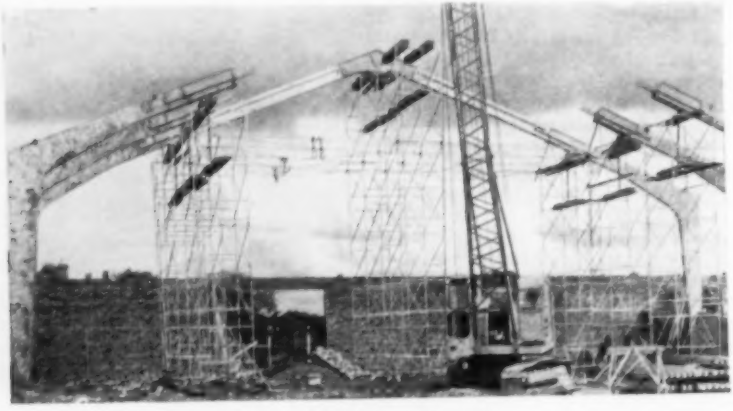

Fig. 3.—Method of Erection.

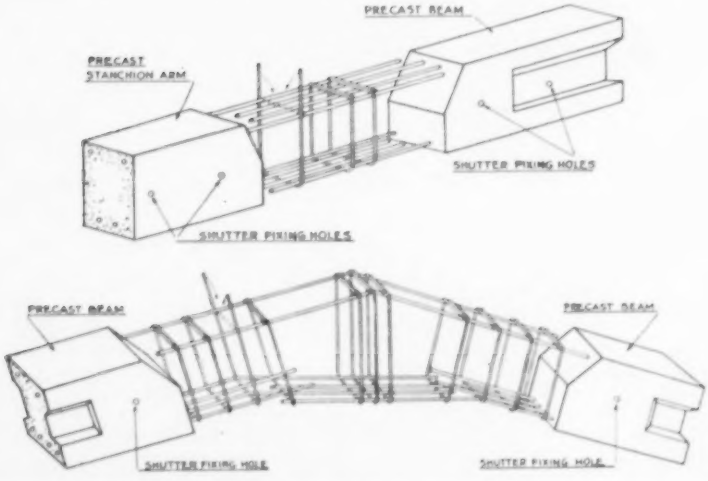

Fig. 4.—Joints between Rafters and Column Members (top) and at Apex.

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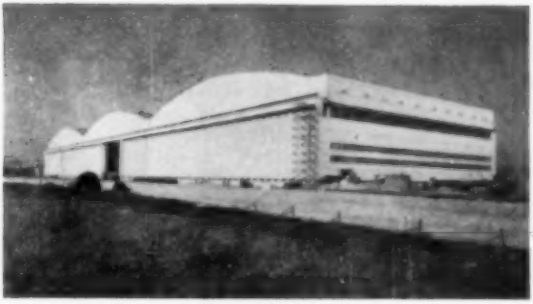

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Fig. 5.—Making the Column Members.

The in-situ joints are made with a 1 : 1 : 2 mixture by volume, using gravel aggregate graded from ⅜ in. down. High-alumina cement is used to obtain high early strength and thus enable the scaffolding to be quickly released. The joints are protected during hardening to reduce shrinkage. All the precast members are made with a 1 : 1½ : 3 mixture with gravel aggregate of ¾ in. maximum size, and are vibrated.

The roof is covered with asbestos-cement sheets and glazing and lined with fire-resistant insulation board. A few openings provided for fans are framed with steel angles, as it would have been uneconomical to make moulds for special purlins. The main walls are of precast concrete hollow blocks which will be rendered and painted with cement externally.

The roof of the boiler house is on prestressed purlins 30 ft. long supported on side walls. Within the factory, various pits up to 14 ft. deep for plant are constructed in normal reinforced concrete. An ancillary building for peeling logs is constructed with precast frames of 30-ft. span.

The design of the factory, the manufacture of the precast members, and the construction have been carried out by the Scottish Construction Co., Ltd., of Edinburgh.

The Strength of Prestressed Beams.

THE following note is taken from the Report of the Building Research Station for the year 1954 (H.M. Stationery Office. Price 4s.).

Studies of the behaviour of prestressed concrete beams under static and dynamic loading have continued on beams with a span of 20 ft. The primary object of the tests has been to determine the resistance to impact of beams made by different methods of prestressing, and to compare their strength with that of ordinary reinforced concrete beams. All the beams were designed for the same static working load and represented normal practice in their design. Although the investigation is not yet complete the following conclusions may be drawn on the impact resistance of prestressed concrete.

(1) With the larger sizes of beam, secondary mild steel reinforcement was required in prestressed concrete members to prevent premature shear failure under impact conditions, and to ensure that the capacity of the member for absorbing energy was fully developed.

(2) Beams with pre-tensioned steel had a higher resistance to shear failure than those with grouted post-tensioned cables or bars; otherwise there was little difference in behaviour between beams stressed by different systems of post-tensioning, but grouting of the steel was necessary to avoid instability during recovery after impact loading.

Heating Systems Installed in Floors.

THE method of warming buildings by means of heating elements embedded in the floors is being increasingly used in residential flats and commercial buildings. These systems operate at much lower surface temperatures than other central-heating systems; for example, hot-water wall-radiators must usually have a mean temperature of 160 deg. Fahr. to produce a mean room temperature of, say, 60 deg. Fahr., whilst a floor-warming system for the same room would have a maximum surface temperature of 75 deg. Fahr. In a floor-warming system the effect is mainly radiant, the warmth is imparted by a surface only 7 deg. to 13 deg. Fahr. higher than the mean room temperature, and the warmth originates at the lowest level. Also, a floor-warming system stores sufficient heat in the fabric of the building to allow the consumption of electricity to be restricted to off-peak hours.

At present, electrical floor-warming installations can be made in three ways, namely, by embedding bare wires directly in the floor for operation at low voltage, by embedding insulated metal-sheathed cables directly in the floor for operation on mains voltage, and by embedding ducts in the floor to carry insulated cables operating on mains voltage.

In the " Panelec " system described in the following the ducts carrying the heating cables are laid in the floor, bonded together to form continuity to earth by means of a bonding strip, and covered with expanded metal or wire mesh to ensure an even distribution of heat. Channels arranged to accommodate the floor finish give access to the connections so that a replacement can be made without disturbing the floor or other parts of the installation.

The ducts are D-shape in cross section, and a layer of expanded metal is incorporated immediately above the tubes to ensure an even distribution of heat (*Fig.* 1). Control is by thermostats operating principally from radiant temperatures as opposed to air temperatures. A domed cover prevents warm air coming into contact with the mechanism, and the efficiency is estimated to be 90 per cent. Thermostats are usually fixed between 5 ft. and 6 ft. above the floor.

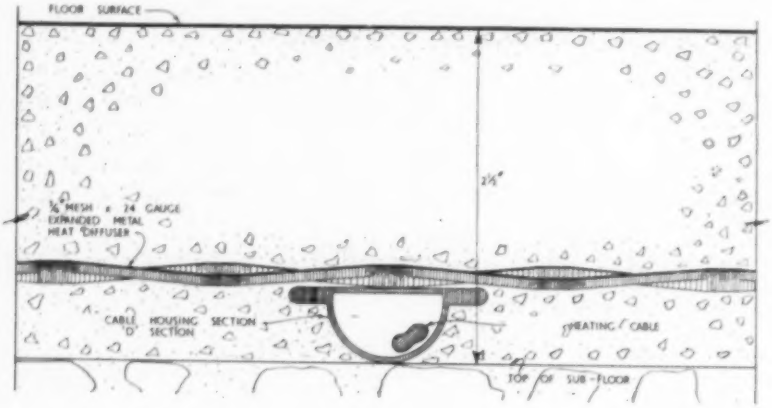

Fig. 1.

Table I gives data relating to various types of structures in which this method of heating has been installed. *Fig.* 2 shows the ducts, cables, and expanded metal being installed, *Fig.* 3 shows the concrete floor topping being placed, and *Fig.* 4 shows a cable being tested.

The Kirkcaldy Corporation has built an eight-story block of all-electric flats as a prototype for a further five blocks.

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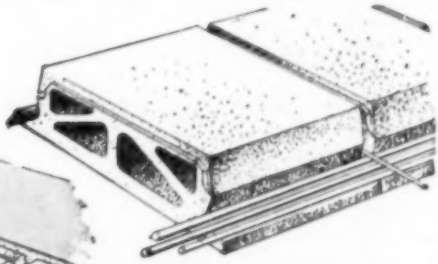

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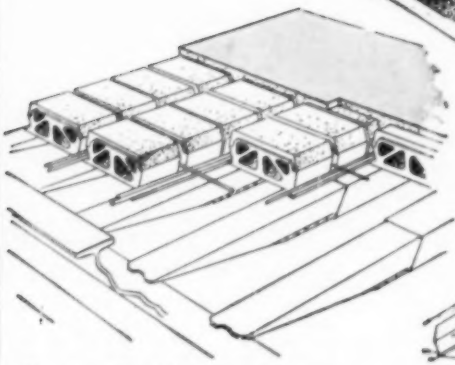

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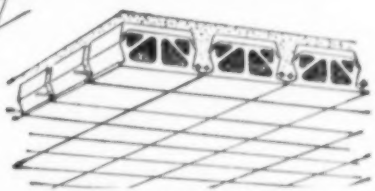

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TABLE I.—Data Relating to Heated Floors.

	Calculated average temperature rise (deg. Fahr.)	Floor area (sq. ft.)	Capacity of building (cu. ft.)	Installed heating load (kW.)	Annual consumption (units)
Public building . .	35	20,000	258,000	262	377,000
Workshop . . .	32	24,000	365,000	286	461,000
Office building . .	32	15,000	185,000	172	237,000
School	30	9,500	94,000	72	*111,000
Offices	32	3,000	24,000	31	49,500

* Includes electricity used for drying-out building.

Fig. 2.—Laying Expanded-metal Heat Diffuser.

Fig. 3.—Placing the Floor Topping.

The structures comprise a reinforced concrete frame with no-fines concrete walls 12 in. thick. The intermediate floors are of precast slabs 5 in. thick, overlaid with 1 in. of paper-bound glass-wool insulation which is compressed to ½ in., and finished with a concrete topping to a depth of 1¾ in. The glass wool is primarily for sound insulation. The undersides of the beams are plastered. In each flat the living-room and the hall will have floor heating designed to provide a temperature of 60 deg. F. and in addition an electric fire is to be fitted in the living-room. It is estimated that the cost of floor heating for a period of 210 days each year will be about £14 per flat.

The warmth is provided by low-temperature radiation, using the floor as the

Fig. 4.—Testing a Cable.

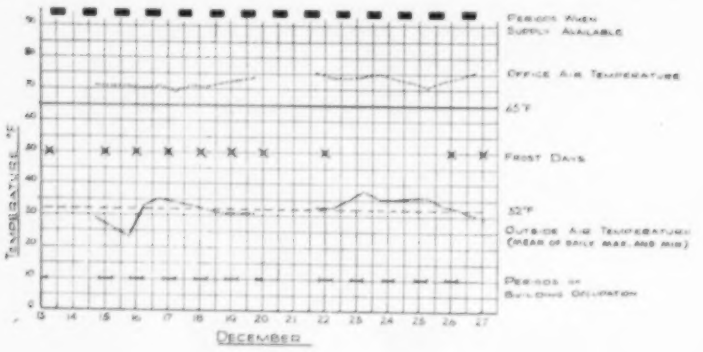

Fig. 5.

transmitting surface. The heating cables are insulated cupro-nickel conductors covered with lead alloy. They are in pairs, but form a single conductor doubled back at its mid-point before it is sheathed; the connections to the electrical supply are therefore made at one end of the cable only. The cables are drawn into aluminium housings designed for maximum transmission laid (in this instance) on top of the insulation and ending at a point where, when the topping and floor finish are applied, there will be access troughs which contain the terminal boxes and facilitate the drawing-in of the cables. On top of the housings expanded metal with ¾-in. mesh is laid and secured to the housing by bonding-plates to act as a heat diffuser. The whole is covered by a topping to a depth of 2½ in. over the beams. The trough-covers are of the removable recessed type fitted into floor registers, and during the laying of the topping these are filled with concrete. The troughs and covers, being flush with the floor surface, are covered with the floor finish.

Fig. 5 shows the results obtained in a floor-heated building of the South of Scotland Electricity Board during exceptionally severe weather. The building had not long been erected and it was overheated in order to assist drying-out. The internal air temperature was maintained at over 70 deg. Fahr., although on some days the average outside temperature was well below freezing.

Loss of Prestress with Time.

In the Report of the Building Research Board for the year 1954 (H.M. Stationery Office. Price 4s.) it is stated that measurements of strain in the prestressed concrete main beams of an office building at Kilburn (described in this journal for March 1954) have continued in order to observe the magnitude of the losses of prestress in the steel. There has been an appreciable increase in this loss during the year due to creep and shrinkage of the concrete. After 2½ years this loss has now increased to about 12 per cent., and is only slightly less than the ultimate loss assumed in the design. The report continues: "Analysis of the behaviour of this structure and of tests in the laboratory shows that the total loss of stress in the steel due to elastic shortening of the concrete on stressing, to creep and shrinkage of the concrete, and to relaxation of the steel may amount to 20 per cent. for post-tensioned steel and possibly to 25 per cent. or more for pre-tensioned steel. These losses are greater than the usual allowance of 15 per cent. made in design. Such under-estimates of the losses increase the risk of cracking, but would be expected to have little influence on the ultimate strength of a structure."

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SITUATIONS VACANT. TWISTEEL REINFORCEMENT, LTD., have vacancies for designers and detailers in their London, Birmingham, and Manchester drawing offices. Apply, stating experience, to 43 Upper Grosvenor Street, London, W.1.

SITUATIONS VACANT. Reinforced concrete designers (senior and junior) required by consulting engineers for varied and interesting building frames and industrial structures. Good prospects. Five-days' week. Lunch vouchers. Apply, with details of experience, etc., to JOHN F. FARQUHARSON & PARTNERS, Chartered Structural Engineers, 34 Queen Anne Street, London, W.1. LANgham 6081.

SITUATIONS VACANT. Reinforced concrete designers and detailers required. Permanent position with good prospects. Pension scheme, five-days' week, and luncheon vouchers. Write or telephone for interview. E. J. COOK & CO. (ENGINEERS), LTD., 54 South Side Clapham Common, London, S.W.4. MACaulay 5522.

SITUATIONS VACANT. Reinforced concrete designer-draughtsmen required in London for varied and interesting work. Sound knowledge of design and practice and ability to produce quick and accurate work essential. Good salaries and permanent positions for right men. For application form apply to SIR BRUCE WHITE, WOLFE BARRY & PARTNERS, 1 Lygon Place, London, S.W.1. Telephone : SLOane 0431.

JOHN LIVERSEDGE & ASSOCIATES, CONSULTING STRUCTURAL ENGINEERS, invite immediate applications for the following additional drawing-office staff.

(*a*) Designer-detailers (3), minimum 3 years' experience, for reinforced concrete work of varied types. Structural steelwork experience an advantage ;

(*b*) Detailer-draughtsmen (6), with experience in reinforced concrete detailing, preferably H.N.C. standard ;

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SITUATION VACANT. The British Reinforced Concrete Engineering Co., Ltd., have a vacancy for a young reinforced concrete engineer to train as a technical representative. Applications to SALES DIRECTOR, B.R.C. ENGINEERING CO., LTD., Stafford.

SITUATION VACANT. W. & C. French, Ltd., have a vacancy in their drawing office at Buckhurst Hill for a reinforced concrete designer interested in work of considerable variety, including reinforced concrete framework, piled foundations, prestressed concrete, and temporary works. Apply in writing, stating age, experience, and salary required, to the PERSONNEL MANAGER, Buckhurst Hill, Essex.

SITUATION VACANT. Senior structural engineer specialising in design of reinforced concrete structures required by Westminster consulting engineers to work in their Surrey office. The position will be permanent and progressive. A self-contained ground floor flat is available to successful applicant if required. A five-days' week and pension scheme are in operation. Apply, stating experience, salary required, and qualifications, to BOX 4356 CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14, Dartmouth Street, London, S.W.1.

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SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., have vacancies for reinforced concrete designers and detailers, with some experience, in their Stafford, London, Liverpool, Bristol, Glasgow, and Newcastle-upon-Tyne offices. Staff pension scheme and five-days' week. Apply in writing to CHIEF ENGINEER, Stafford.

SITUATIONS VACANT. Westminster consulting engineers require immediately for their London office structural (reinforced concrete) assistants, preferably with experience in sewerage, sewage disposal and water supply. Must be fully conversant with design of water-retaining structures. Salary according to qualifications and experience. Pension scheme. Luncheon vouchers. Apply, with full particulars and with at least two references, to J. D. & D. M. WATSON, 18 Queen Anne's Gate, Westminster, S.W.1.

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SITUATIONS VACANT. Consulting structural engineers, Westminster, require first-class designer-draughtsman and a detailer experienced in reinforced concrete and/or steel framed buildings and foundations. High salaries and good prospects for suitable applicants. Write in confidence stating age, qualifications, and details of experience. BOX 4167, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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Starting salary (up to £1100 a year) will depend on background. The men appointed will have the opportunity of visiting construction sites, and working conditions are good. The posts are pensionable. Please send brief relevant details (quoting ref. CN49) to Staff and Training Division, Simon-Carves, Ltd., Cheadle Heath, Stockport.

SITUATIONS VACANT. Two civil or structural engineers required in firm of factory consultants. Applicants must be competent draughtsmen, of Higher National Certificate standard, with some knowledge of design. University degree, or equivalent, with little experience, equally preferable. Salary £600-£700 per annum, plus bonus. Good prospects, including professional experience necessary to qualify for A.M.I.C.E. or A.M.I.Struct.E. Staff pension scheme. Offices near main line terminus. Applicants should telephone CHIEF ENGINEER in the first instance for an appointment (Paddington 9203).

SITUATIONS VACANT. Reinforced concrete engineers require designers, detailers, and draughtsmen at their Keynsham (Bristol) office. Vacancies offer opportunity to work on a variety of structures, and salaries will be dependant on age, experience, etc. Previous similar experience an advantage, but consideration will be given to other suitable applicants. BOX 4169, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Civil engineering contractors require young structural engineer with specialist experience in precast concrete construction. Some architectural experience an advantage. Apply, giving full particulars, to MEARS BROS. (CONTRACTORS), LTD., Harbour Road, Rye, Sussex.

SITUATIONS VACANT. Designer-draughtsmen required for London head office of old-established reinforced concrete engineers. Applicants must be familiar with design of reinforced concrete frames and all forms of reinforced concrete floor construction. The post is permanent, progressive, and pensionable. Existing holiday arrangements would be respected. Substantial salary for experienced men. Write BOX 4170, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. THE STANTON IRONWORKS CO, LTD., near Nottingham, invites applications for the appointment of assistant works manager on its concrete plant. Applicants should have had a good general engineering

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The post will be superannuable, and the successful candidate may be required to undergo medical examination.

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training, including a knowledge of precast concrete construction and preferably have held a similar position in the precast concrete products industry. A knowledge of prestressed concrete would be an advantage. The post is superannuated. Write in confidence, giving age and full details of qualifications and experience, to the STAFFING OFFICER.

SITUATIONS VACANT. Draughtsmen. Imperial Smelting Corporation, Ltd., Avonmouth, Bristol, requires experienced mechanical, electrical, and structural/civil draughtsmen for design and detail work in connection with chemical metallurgical plant and equipment. Salaries, working conditions, and welfare facilities are good, and good prospects of promotion exist for young men holding or prepared to obtain H.N.C. or equivalent. Bus and train services to and from the works gates. Write to PERSONNEL MANAGER for application form, giving brief details of age, qualifications, and experience, quoting reference D/CCE.

(Continued on next page)

MISCELLANEOUS ADVERTISEMENTS.

(Continued from previous page.)

SITUATIONS VACANT. Civil and structural engineering assistants experienced in reinforced concrete and structural steel required. Knowledge of design will be an advantage but not essential, depending on positions. The work is varied and offers scope for initiative. Five-days' week, pension scheme. Good salaries will be offered to suitable applicants who should state experience and salary required, to ENGINEERING DEPARTMENT, FARMER AND DARK, Romney House, Tufton Street, London, S.W.1.

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SITUATIONS VACANT. CLARKE, NICHOLLS & MARCEL, consulting engineers, have vacancies in their London and provincial offices for reinforced concrete designer and detailing draughtsmen. Salary and advancement dependent on experience and capabilities. Reply to 21 Westbourne Grove, London, W.2.

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SITUATIONS VACANT. Senior reinforced concrete designers wanted by leading reinforced concrete engineers and contractors. Must be fully conversant with Code of Practice, L.C.C. Bye-Laws, and able to design light-framed structures from estimating stage to final details. Five-days' week. Pension scheme. Progressive position. Starting salary from £900 upwards according to ability. This year's holiday arrangements honoured. Juniors also required, similar conditions. Write BOX 4172, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Detailing draughtsmen and designers for reinforced concrete or steelwork required. Good experience in varied work. Five-days' week and bonus. Apply in writing, giving particulars of experience, and salary required, to T. F. BURNS & PARTNERS, 25, Victoria Street, London, S.W.1.

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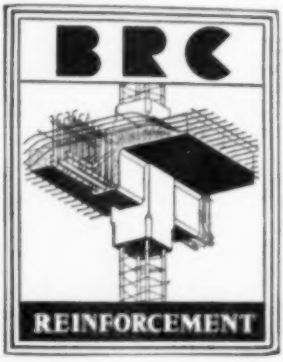

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